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CONTENTS

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ON CUTOFF PROCESS OF THE RIVER ARIAL KHAN

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Abstract

In a meandering river, the length of the river keeps on increasing by eroding the outer bank of the bend. This increase in length will continue if any counterbalancing mechanism does not come into play. In reality such ever increasing length of the river is shortened by a mechanism called cutoff. The extent of meandering before a cutoff occurs is generally characterized by a parameter called cutoff ratio. Cutoff of a river bend may occur naturally or it can be man made. In order to study the morphological behavior of a river it is imperative to develop understanding of the parameters that influence the cutoff process of that river. Besides it is also important to have a time scale to predict when a cutoff may occur. In this study attempts have been made to recognize the cutoff process of the river Arial Khan. A method is also developed to predict the time required for cutting a certain cord length off. The study shows that the local resistance largely influences the cutoff of the river Arial Khan and cutoff occurs at a very high value of cutoff ratio.

Introduction

When the length of a particular bend exceeds a certain value the bend will be cutoff by the scouring of a cutoff channel in the floodplain through the neck of the meander. An oxbow lake will remain, and this lake will be silted up gradually. A meandering river has a shallow side channel together with its main curvilinear path. This might be the remains of its old course or may be created by floods spilling over the river bank. During high floods, excessive deepening of the pools occur, and is supplemented by the growth of bars at the inflections. Both these factors tend the water to flow more and more towards the side channel. When the flow starts reducing in the main channel, more and more silting occurs in the main channel, which further increases the flow in the side channel. This process continues and finally a time may come when the entire water starts flowing from the full developed chord channel and the curvilinear path gets silted up. The period, over which the opening of a chord channel matures into a full-fledged cutoff may vary considerably (Garg, 1991). The two essential criteria for a cutoff to occur are: (1) The shear stress should exceed the critical shear stress and (2) The sediment entering the potential cutoff channel should be less than the sediment transport capacity of the cutoff channel.

Cutoff ratio is a parameter that characterizes the extent of meandering before a cutoff occurs. It is defined as the ratio of the length of the bend to that of the chord (Figure-1). It is stated in Joglekar (1971) that " this ratio varies depending on the characteristics of the river at site, such as the discharge, the river flood stage, surface fall, bed material and its suitability for the growth of protective grass and weeds". Some numerical values

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for the cutoff ratio are given in the same publication and they vary between 1.7 for Punjabi rivers in India and Pakistan and 8 to 10 for the Mississippi River.

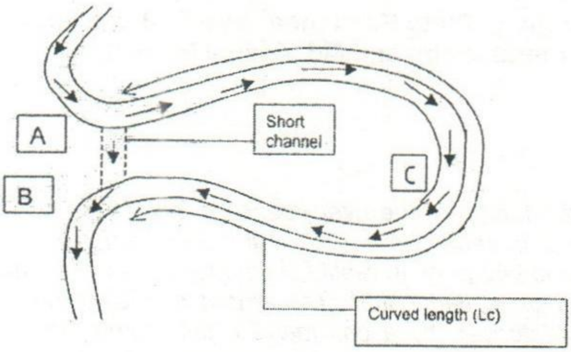

Figure-1: Definition of cutoff ratio

The river Arial Khan is no exception to the other rivers of Bangladesh, which carry huge amount of sediment load along with water during monsoon. Again during lean flows,, there is heavy deposition of silts. These altogether cause significant geomorphological change that is required to be assessed. In this context, the present study is undertaken to develop understanding of the parameters which influence the cutoff process and to evolve a method by use of which time required for cutting a certain cord length off can be predicted.

About The River Arial Khan

The river Arial Khan is one of the main distributaries of the Ganges flowing through the deltaic plains. The river has two parts, one of them is called as the Arial Khan Upper and the other is the Arial Khan. Both the parts of the river take off from the right bank of the Ganges near Chowdhury Char and Dubaldia respectively. The Arial Khan is a meandering river. The length of the Arial Khan Upper and Arial Khan is about 70 Km and 125 Km respectively. The distance between the Arial Khan Upper at their junctions with the Ganges is about 30 Km and the distance between offtake of the Arial Khan Upper and Aricha confluence is about 70 Km.

The Arial Khan is navigable year-round and almost throughout the whole river course. This navigation facility is generally used by the surrounding inhabitants for their day to day travel to the nearby market place and transporting of commercial commodities and agricultural products to other trade centers of the country.The historical evidence and the evidence of the recent years reveal that formation of sharp bends and loop-cut is a characteristic feature of the river. In the past the river had undergone tremendous planform changes through this process and the process is still very much active. It is also clear from the available evidence that the Arial Khan mouth is not at all stable in its position. Many oxbow lakes, e.g., the traces of the previous meanders that have been cut off are noticeable from the satellite images of different years.

Literature Review

It is noticed from Garg (1991) that a cutoff may be developed, if the following conditions are satisfied:

1. The cutoff ratio varies from 1.7 to 3.0 or more.
2. The ratio $r/(Q_{max})^{0.5}$ is between 13 to 24, where r is the radius of curvature and Q_{max} is the maximum discharge. A cutoff formation is accelerated if the curvature is too sharp for discharge.
3. The shallow side channel is tangential to the main direction of river flow approaching and leaving the cut.

Klaassen and Zanten (1989) made an theoretical analysis considering a schematized river bend which may help to get a better understanding of the parameters that influence the cutting off process and that determine the value at which cutting off take place. Their analysis is given initially for a simplified and later for a more complete case. Both cases considered deal with the initial erosion. In the first case it is assumed that although the river flow is above bankfull condition, no water is flowing perpendicular to the curved channel except the channel formed along the cord and also that the upstream water level is not affected by the discharge flowing over the floodplain. From this analysis some remarks were made which only holds for the beginning of the development of the cutoff channel. The remarks are:

1. The larger the value of the cutoff ratio, the more probable the development of a cutoff.
2. The larger the flood, the more bends will be cutoff
3. $\eta\ (\lambda)^{(n-1)/(n-3)} > 1$, where η is defined as the ratio between initial water depth of the short channel to that of the curved channel and λ is cutoff ratio. The value of n varies between 3 to 10. The smaller the value of n the easier cutoffs occur.

In the second case of the analysis the assumption that the ratio between short channel width to curved channel width (β) is small was dropped which allows for a drop in water level at the bifurcation. However, it was still assumed that the drop in the water level is small compared to the water depth present at the bifurcation and that in both channels normal depth is present at the bifurcation. Moreover, the following assumptions were made for this:

$C_{short\ ch.} = \gamma C_{curved\ ch.}$ and

$$\frac{S_{short\,ch.}(\text{incoming})}{S_{curved\,ch.}(\text{incoming})} = \frac{1}{\sigma}\frac{Q_{short\ ch.}}{Q_{curved\ ch.}}$$

The value of σ depends on the geometry of the bifurcation, on the dominant mode of the sediment transport, and probably on the value of η.

The following observations were made from the analysis:

1. The larger the value of σ, the easier a cutoff will take place. This implies that in rivers where bed load is dominant over suspended load the sinuosity will be less, provided that all other factors are the same.
2. The effect of β is substantial. It can be stated that an increase of the width of the cutoff channel results in more stable conditions.

The authors also obtained a time scale for cutoff and remarked that this time scale for the cutoff process is a very crude one.

Data Acquisition

The data required for the study were maps, satellite and spot images and field survey information. Maps were collected from SOB (Survey of Bangladesh) and satellite and spot images were collected from SPARRSO.

Methodology

The collected maps and images for the years ranging from 1985 to 1998 had been prepared at scale of 1:50,000 and this scale was found suitable for analysis. River bend displacements had been measured by superimposing maps and images of different years and matched by using common water bodies located as near as possible to the plane of the river floodplain and more or less fixed in time and space. The changes in the curved length, cord length etc. over the time period considered had been determined for all bends throughout the entire reach of the river. The extracted data were then analyzed to show the different aspects of the cut-off processes of the river.

Cutoff Process in the Arial Khan

In order to understand the cutoff process of the river Arial Khan information available from the maps and satellite images have been analysed and also field visit is made. In the period for which information is available several natural cutoff are found to occur. But due to long time interval between the maps it is not possible in all cases to recognize the process by which those cutoffs took place. In all recognizable cases it is noticeable that immediately after the cutoff a new loop tends to develop downstream of the cutoff.

In the Arial Khan it is observed that the local resistance to the formation of a cutoff channel is very high. In the study period a fully developed meander was found immediately upstream of the Arial Khan Ferry Ghat on Bhanga-Mawa Highway. From the satellite image it is found that in 1998 situation the cutoff ratio was as high as 13. In 1999, the neck length was shortened further to 274 meter indicating substantial increase in the cutoff ratio. From the recent field visit it is seen that the cord channel is fully opened but full-fledged cutoff channel is yet to develop. It may take one or more years. It is also evident from this real situation that the cutoff channel starts to develop from the downstream side. It has happened due to the fact that at the downstream side of the cutoff channel, velocities are highest owing to the drawdown that may occur. It is to be noted here that the above mentioned meander started to develop immediately downstream of a meander which was cut off after 1967. It is, therefore, clear that the life time of this meander is about 25 years. However, it does not apply to all meanders of the river.

Regarding cutoff ratio at which cutoff occurred in the river some other observations have also been made. In one observations it is found that cutoff occurred at a cutoff ratio 17.3. In another observation it is revealed that in 1985 situation cutoff ratio was 11.5. In 1988 situation a cord channel is seen to be developed there. No information is available as to the situation in the following years. However, it can be fairly imagined that cutoff in this meander occurred at a cutoff ratio of 11.5 or more.

In meandering rivers cutoff channels develop usually in the floodplains. Cohesive soils and vegetation often characterize these floodplains. Both aspects have a pronounced influence on the resistance to erodibility. Hence often those cutoffs do not develop easily. For meandering rivers cutoff ratios are found to vary between 5 to 30. Since in the river Arial Khan the above-mentioned characteristics are very much pronounced, the cutoff ratio's found at the cutoff are quite realistic.

It is obvious that the cutoff occurs when a bend reaches its maturity. In nature, as the bend develops the cord length gets shortened and ultimately reduces to such a short length where cutoff becomes possible. In this present study an attempt is made to analyze this process by use of available information. Three representative bends are selected for the analysis. The average cord length, curved length and cutoff ratio have been determined for the year of 1985, 1988, 1992, 1995 and 1998. The results appear in the following table (**Table-1**)

Table-1: Average cord length, curved length and cutoff ratio for different years

Year	Curved length (Km)	Cord length (Km)	Cutoff ratio
1985	4.252	1.79	2.53
1988	5.17	1.63	3.16
1992	5.88	1.40	4.21
1995	6.37	1.04	6.10
1998	6.58	0.86	7.66

The information presented in the table shows that curved length increases, cord length decreases and cutoff ratio of bend increases with time, which is obvious. But what are important to notice from the information are as follows:

1. The rate of increase of curved length reduces with time.
2. The rate of shortening of cord length increases with time upto a state from where cutoff can occur within a short time depending on the occurrence of a flood having magnitude and duration enough for cutting the cord channel off. Above information suggests that during such a critical cord length the rate of shortening of the cord length is somewhat lesser than that what it experienced in the previous time period. It seems that in this state the river concentrates more on eroding the bed of the cutoff channel.
3. Initially % increase in the curved length is higher than the percent increase in the cord length reduction. But with time as the cutoff ratio increases percent increase in the cord length reduction exceeds % increase in the curved length.
4. Over the timespan considered the yearly rate of increase of curved length is about 158 m and the yearly rate of cord length reduction is about 71.7 m.
5. Over the timespan considered it can be seen that the curved length is increased by 45.5% whereas the cord length is reduced by 52%.

An examination is made to notice the influence of discharge magnitude on formation of cutoff. It is done for the Arial Khan Upper. It appears from the examination that cutoff formation has no direct relationship with the discharge magnitude. In most of the cases it is seen that a moderately high discharge is able to cause as much erosion as a very high discharge. It will not be out of place to mention here that a natural cutoff occurred immediately upstream of the Arial Khan Ferry Ghat after the year 1976 which is visible in the satellite image of 1980. It can be seen from the discharge data that in the time period between 1976 to 1980, both the annual maximum and annual average discharge of the Arial Khan were of very low magnitudes compared to the recorded ever highest discharges of the same. However, it does not imply that discharge magnitudes do not have any influence on the cutoff formation. The important factor in this regard is the duration of the flood.

Prediction of time for cutoff

In a meandering river development of meander loops and cutoffs is a natural phenomenon. If a meander begins to develop it can be stated that once the bend will be matured and then will be abandoned by cutoff. It is therefore, necessary to have a time scale to predict when that cutoff will occur. So far, no such reliable time scale has been developed. If it becomes possible to develop such a time scale the question then may arise about its universal applicability. Within the framework of this study efforts have been put to develop a method that enables to predict a time range within which cutoff of a meander having certain cord length will occur. The method is developed by use of the data of the Arial Khan and therefore applicable for the Arial Khan only. In order to develop the method four bends were identified where cutoff occurred in the period between 1944 and 2000. For individual bend cord lengths have been measured from the time of cutoff to the time when the bend started to develop from the collected satellite images. In this way a set of data is obtained for four separate bends to plot a number of years (N) necessary for cutoff versus cord length graph. In the graph three linear trendlines are shown (Figure-2). Among the three trendlines two represent extreme situations and one gives an average impression of time required for a cutoff to occur by cutting a certain cord length off. Although in nature everything is very much unpredictable and a linear relationship can never exist, the method can still be acceptable in view of the existing reality in order to have an impression of time scale for

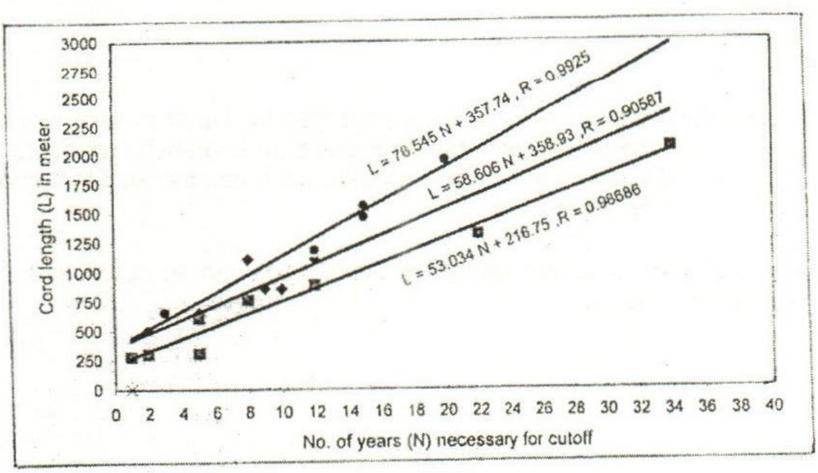

Figure-2: Cord length (m) versus number of years necessary for cutoff

cutoff. In the method developed for predicting time scale the extreme as well as the worst case is represented by the following equation:

$$L = 76.545\ N + 357.74$$

Where, L (m) stands for the cord length that is to be cut off and N (years) stands for time required for cutting the L off. As mentioned above this equation gives a minimum time for a cutoff to take place.

The other extreme situation, i.e. maximum time required for cutting a certain cord length off is given by the following equation:

$$L = 53.03\ N + 216.75$$

On the other hand the following equation gives an average impression of time required for cutting a certain crude length off:

$$L = 58.606\ N + 358.93$$

Now verification is made for the above equations for a observed field situation. In case of a meander where the cutoff channel is fully opened in 2000 and it is expected that a full-fledged cutoff channel will be developed by the year 2001. In 1992 situation the cord length of this bend is seen to be 830 m. From the above equations three different times will be obtained for this development of a full-fledged cutoff channel by cutting this cord length of 830 m off which can be compared with what happened in reality. These three different times are given below:

$N_{max.} = 11.5$ years

$N_{min.} = 6.2$ years and

$N_{av.} = 8.0$ years

Thus from the equations a time range can be predicted for cutoff to take place which ranges from 6.2 to 11.5 years. The equation that gives an impression of average time yields 8 years for a cutoff to occur in the case considered. It can be seen that in reality it takes about 9 years to complete the cutoff.

An interesting observation is made regarding the cord length at cutoff and channel width, which is presented below:

Table-2: Observed channel width and the cord length at cutoff

Year	Location of the bend	Channel width (Km)	Cord length at cutoff (Km)
1999	Immediately upstream of the Arial Khan Ferry Ghat	$\cong 0.294$	0.274
1976	Upstream of the above bend	0.308	0.28
After 1985	Downstream of the Arial Khan (Upper) offtake	$\cong 0.319$	< 0.47

The information shown in the above table gives an impression that the cutoffs generally occur at cord length not much narrower than the channel width.

Discussions

From the maps and satellite images available for the time period between 1944 and 1988 several development of natural cutoffs are noticeable. In the previous sections a number of practical conclusions are drawn concerning meander development and cutoff. It is quite clear from the observations that every cutoff influences channel development on the downstream reach. The reason behind this is that as a result of the cutoffs, the water surface slope and the flow velocity increase locally causing additional erosion of the banks on the downstream reaches. It can be fairly predicted from the above mentioned facts that the downstream reach of the recently formed cutoff immediately upstream of the Arial Khan Ferry Ghat will cause downstream bank erosion. If this erosion is allowed to continue once the Arial Khan Ferry Ghat and also the Dhaka-Mawa Highway will be endangered.

Regarding cutoff ratio it is quite clear that in the Arial Khan cutoff occurs at a very high ratio. In one case it is seen that cutoff was not occurred even though cutoff ratio reached a value of 13. But occurrence of such a high cutoff ratio is not at all abnormal. It can be stated that in the river Arial Khan the local resistance to cutoff is very high. Grain size analysis of soils collected from a recently opened cutoff channel confirms the presence of stiff clayey layer on the channel bed. It is seen that after the primary cutoff, the meander to be abandoned can develop further until the new channel achieves a sufficient width and depth.

Conclusions

- Based on the outcomes of the study the following conclusions have been drawn:
- Generally cutoff occurs in the Arial Khan at a high value of cutoff ratio and local resistance to the cutoff formation is very high.
- Cutoff ratio increases with time and as the cutoff ratio increases the rate of shortening of neck length increases.

- When analysed for a certain time span it is seen that the yearly rate of increase of curved length is about 158 m and yearly rate of neck length reduction is about 71.7 m.
- When cutoff ratio increases to a value where cutoff becomes possible a moderate flood discharge is enough to cause such a cutoff.
- Discharge magnitudes do not have any direct relationship either with the cutoff formation or with the rates of shortening of the neck length.
- It is seen that cutoff occurs at cord length not much narrower than the channel width.
- Development of cutoff channel starts from the downstream side

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A STUDY ON THE GEOMETRIC CHARACTERISTICS AND BANKLINE SHIFTING OF PADMA RIVER

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Abstract

A study was conducted by the River Research Institute (RRI) to investigate the geometric characteristics and bankline shifting of Padma river from Goalanda to Mawa covering a reach of about 60 km. The study includes analysis of river geometry in terms of cross-sectional area, average depth, top width, thalweg level (deepest point) & mean bed level (MBL), development of non-dimensional correlation, variation of stage & discharge, specific gauge analysis and determination bankline movement using satellite images. The study showed that the geometry of the river has undergone considerable change over a period from 1992 to 2000. The study also showed that various amount of bankline movement of Padma river had been occurred at the selected locations in 2001 with respect to 1973 within the study reach.

Introduction

During monsoon, rivers of Bangladesh carry huge amount of sediment along with water. They altogether cause significant change to geomorphic and other hydraulic characteristics of the rivers. Again during lean flows, there is heavy deposition of silts. Thus, throughout the year, changes in cross-section, slope, thalweg etc. of the river are taking place. Again these parameters undergo great changes from year to year due to the change in discharge. The Padma is no exception to these changes.

Keeping this fact in mind a study on the geometric characteristics and bankline shifting of Padma river is extremely important in connection with the river bank stabilization, navigation, flood control and development of different water resources projects in Bangladesh. The location of the study area from Goalanda to Mawa is shown in **Figure 1.**

The Padma river

The combined flows of Jamuna and Ganges rivers constitute the flow of the present Padma river. Before the avulsion of Jamuna river, the flow was a continuation of Ganges river only. The annual mean discharge is 28,000 m^3/s, and the bankfull discharge is about 75,000 m^3/s. The average size of the bed material is about 0.10mm.

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Geomorphologically, the river is still young. A flow regime analysis by FAP4 (1993) shows that it is now in a dynamic equilibrium. The planform of the river is a combination of the meandering and braiding type, indicating a wandering river. The variation of the total width of the river is quite high, ranging from 3.5km to 15km. The slope of the rivers is varying within a range of 8.5cm to 5cm per km. The bed material sizes are also varying from 0.20mm to 0.10mm. The dominant discharge is about 80,000m^3/s, which is slightly lower than that of the bankfull discharge of 82,500m^3/s.

Figure 1 The Padma River from Goalanda to Mawa in 2001(EGIS, 2001)

Methodology

All the available standard BWDB cross-sections were considered in the present study. These were P0, P0.1, P1.1, P2, P2.1, P3, P3.1, P4, P4.1, P5, P5.1, P6, P6.1 and P7. The spacing between successive cross-sections was 6.4 km. The location map showing these cross-sections is shown in **Figure 2.** The cross-sectional area of each cross-section in 1992 was compared with that of 2000.

The thalweg level is the deepest point of the cross-section and was determined for 1992 and 2000 to observe its variation.

The mean bed level (MBL) was determined by subtracting the average depth from the average bank level of each cross-section for 1992 and 2000 to observe its variation.

The relationships of hydraulic geometric parameters such as water width (W), water depth (D), flow velocity (V) and water area (A) against discharge (Q) for the years 1981, 1983, 1984, 1988 & 1992 were developed using monthly average data at Baruria Transit.

The use of non-dimensional correlations in the study of alluvial river geometry is becoming gradually popular for many reasons. One potential application of such correlation is the study of scale models (Hossain *et al.*, 1996). Keeping this in mind, correlations between Froude Number $[V/(gD)^{1/2}]$ & shape [W/D] and non-dimensional discharge [Q/(kD)] and Froude Number $[V/(gD)^{1/2}]$ were established. The trend of each relationship was also determined statistically. The average slope of the river was taken to be 5.0 cm/km and the value of acceleration due to gravity was taken to be 9.81 m/sec^2. Here k is the kinematic viscosity (m^2/s) of water and Q, W & D are water discharge (m^3/s), width (m) & average depth (m) respectively.

The Baruria Transit (Water level plus discharge station, Code No. 91.9L) is the immediate upstream of Goalanda of the Padma river. At Baruria Transit the trend of water level and discharge against time was determined separately using annual average data. The rating curve was also developed using water level and discharge data from 1966 to 1994.

At Goalanda Transit (Water level station, Code No. 91.9R) the trend of water level was determined using annual average data.

At Bhagyakul (Water level station, Code No. 93.4L) the trend of tidal water level (average value taken during ebb tide & neap tide) was determined using annual average data. The location map of the stations mentioned above of Padma river is shown in **Figure 3.**

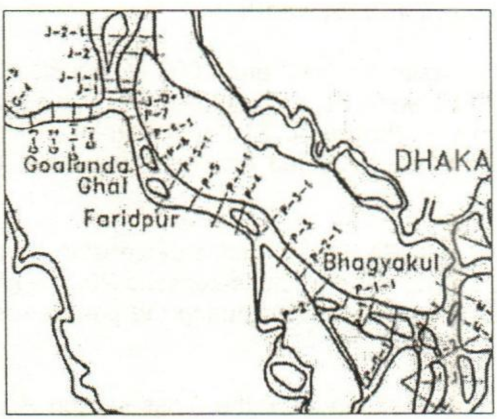


Figure 2 Standard BWDB cross-sections (FAP24, 1996) of Padma river

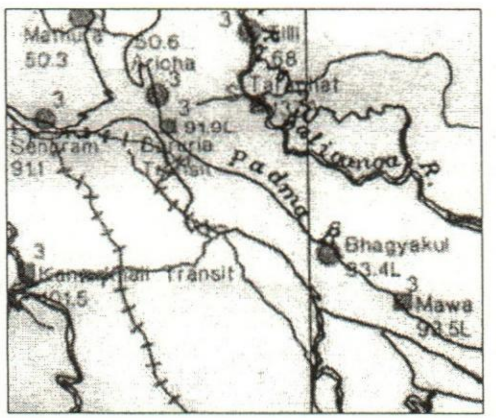


Figure 3 Location map of the hydrometric stations (SWMC, 1996) of Padma river

At Mawa (Water level plus discharge station, Code No. 93.5L) the trend of tidal water level (average value taken during ebb tide & neap tide) and tidal discharge was determined separately using annual average data. The rating curve was also developed using discharge & water level data both for ebb tide and neap tide from 1968 to 1994.

For the analysis of medium to long term behavior of the river, specific gauge analysis was used to determine the trend of stage with time corresponding to a given discharge. This analysis is based on historical stage-discharge records for a gauging station with an open river section (Das, 1992).

Specific gauge analysis was done at Baruria Transit using data from 1966 to 1993 at various discharge levels of 5000, 10000, 20000, 35000 and 50000 m^3/s. These were plotted to show the variation of stage at various discharge levels in various years mentioned above at this station. The trend of water level for each discharge level was then determined.

From the satellite images, the bankline of 1973 & 2001 was superimposed in order to determine the bankline movement of 2001 with respect to 1973. For this purpose five representative sections were selected to represent the study area. These sections are: (i) about 5.0 km downstream of Daulatdia Ghat (ii) about 21.0 km downstream of Daulatdia Ghat (iii) about 34.0 km downstream of Daulatdia Ghat (iv) about 46.0 km downstream of Daulatdia Ghat and (v) about 57.0 km downstream of Daulatdia Ghat. The amount of bankline movement relative to 1973 was determined along the representative sections.

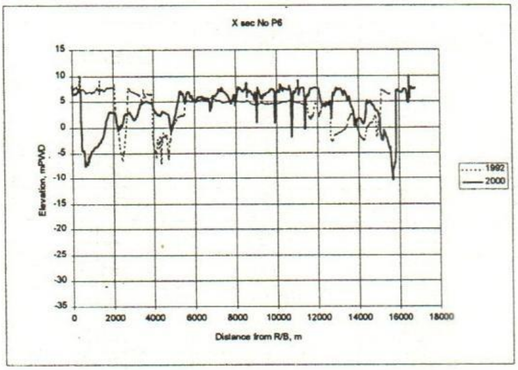
Results and discussion

Variation of cross-sectional area, average depth and top width

Each of the cross-sectional maps was superimposed for 1992 and 2000, which clearly shows the shifting of thalweg, change of width as well as the location of the thalweg. A typical cross-section near Goalanda is shown in **Figure 4.** The variation of cross-sectional area, average depth and top width of the selected cross-sections area is shown in **Figure 5, 6** & **7**.

The cross-sectional area at cross-sections P0, P3, P4 & P5.1 had a decreasing trend indicating aggradation while there had an increasing trend at cross-sections P0.1, P1.1, P2, P2.1, P3.1, P4.1, P5, P6, P6.1 & P7 indicating degradation during the period from 1992 to 2000.

The maximum change in cross-sectional area was observed at the cross-section P6.1 where the cross-sectional area in 2000 was increased to 1.34 times the corresponding cross-sectional area in 1992. The minimum change in cross-sectional area was observed at the cross-section P6. The total cross-sectional areas calculated were 1062966 m^2 and 1113218 m^2 in 1992 and 2000 respectively indicating overall erosion in 2000 with respect to 1992. The net increase in cross-sectional area was about 4.7%. The average depth in 2000 is less than that of 1992 while the average width increases.



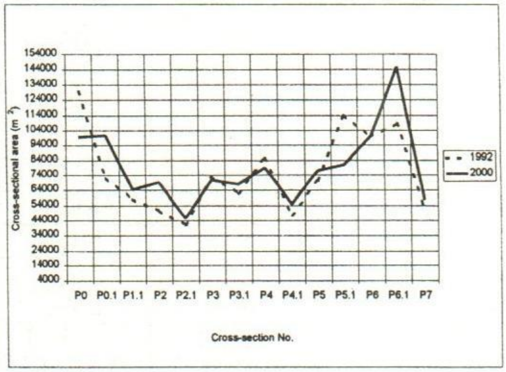
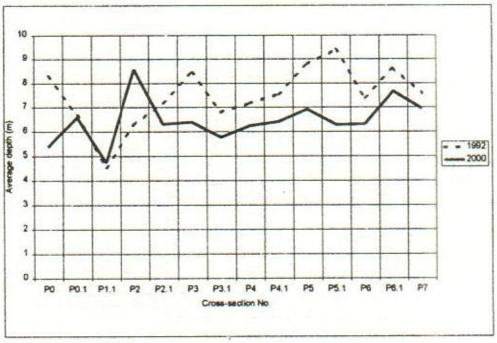


Fig. 4 Superimposed cross-sectional map between 1992 and 2000 (P6)

Fig. 5 Variation of cross-sectional area for the standard BWDB cross sections



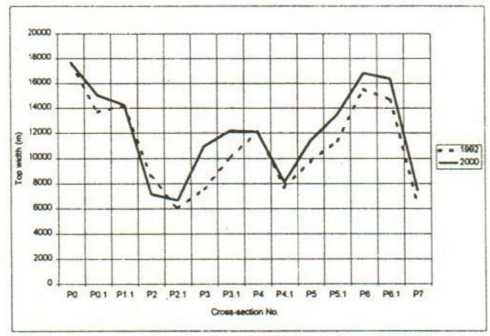


Fig. 6 Variation of average depth for the standard BWDB cross-sections

Fig.7 Variation of top width for the standard BWDB cross-sections

Variation of thalweg level

The position of thalweg measured from left or right bank of river for the cross-sections over a period of 1992 to 2000 is shown in **Table 1**. The thalweg of cross-section P0, P1.1, P2, P2.1, P3, P4.1 & P6.1 was shifted to the right whereas the thalweg of P0.1, P3.1, P4, P5, P5.1, P6 & P7 was shifted to the left. The variation of thalweg level for the standard BWDB cross-sections is shown in **Figure 8**. An exceptional deepest point was found in P0.1, which was 30.4 m below the PWD datum in 2000.

Table 1 Location of thalweg measured from left or right bank for the standard BWDB cross-sections

BWDB C/S	1992 m	2000 M	Measured from	Shifting m	Direction
P0	15914	14648	RB	1266	right
P0.1	594	1772	RB	-1178	left
P1.1	2949	8230	LB	-5281	right
P2	1849	3128	LB	-1279	right
P2.1	848	5037	LB	-4189	right
P3	720	3030	LB	-2310	right
P3.1	9134	2590	LB	6544	left
P4	3850	11054.6	RB	-7205	left
P4.1	6725	4494.49	RB	2230	right
P5	9293	5053.04	LB	4240	left
P5.1	11024	12530.53	RB	-1506	left
P6	4390	15684.52	RB	-11295	left
P6.1	6294	3539.73	RB	2754	right
P7	305	6174.13	RB	-5870	left

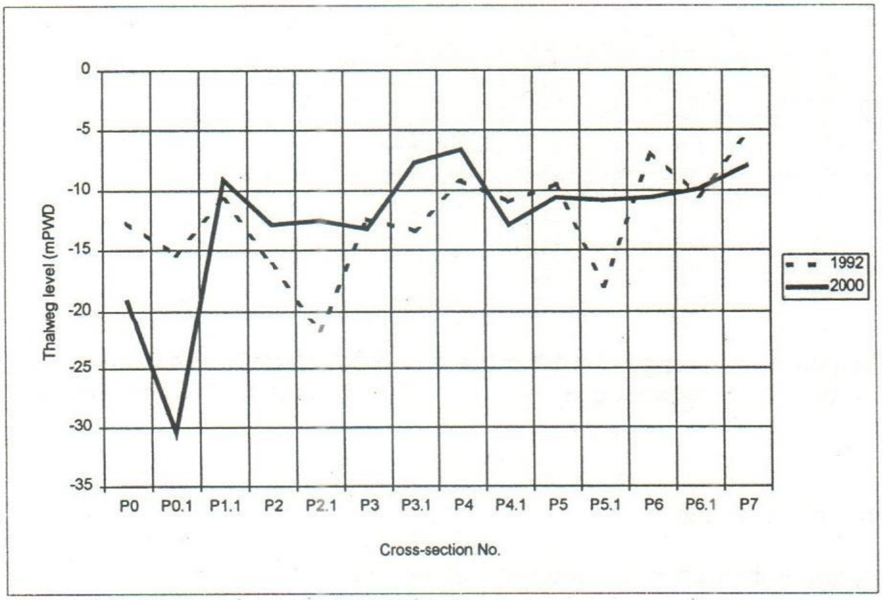


Figure 8 Variation of thalweg level for the standard BWDB cross-sections

Variation of mean bed level (MBL)

The variation of MBL is shown in **Figure 9**. It was observed from the figure that the sediment was deposited at all cross-sections within the study reach except P0, P1.1 & P2. The maximum bed level rises at cross-section P5.1 that was about 3.4 m.

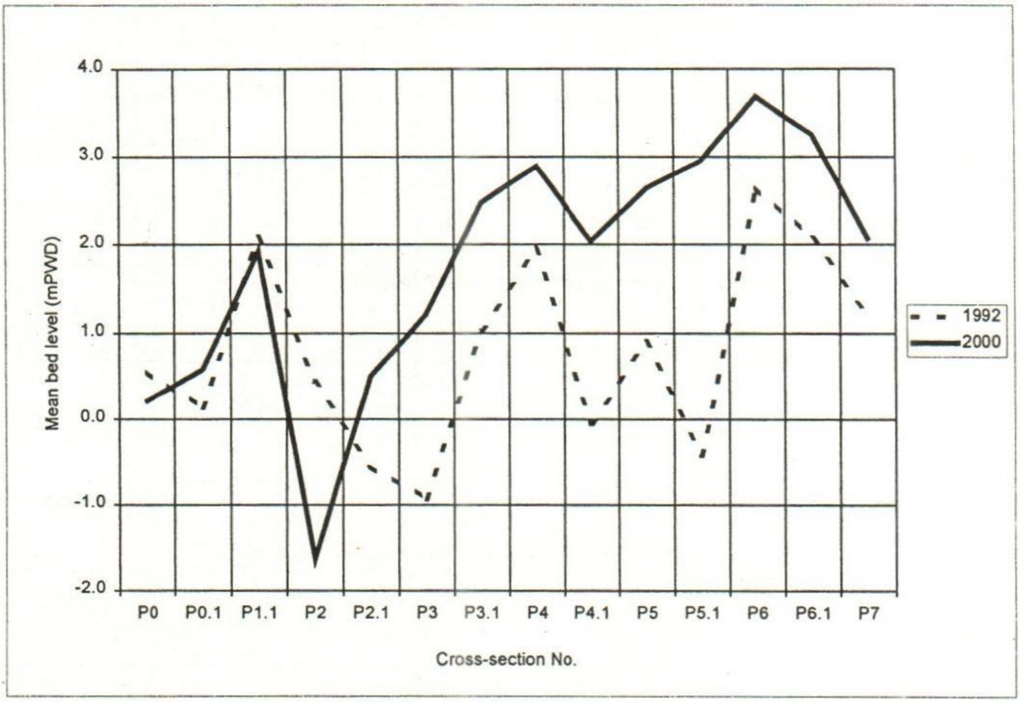
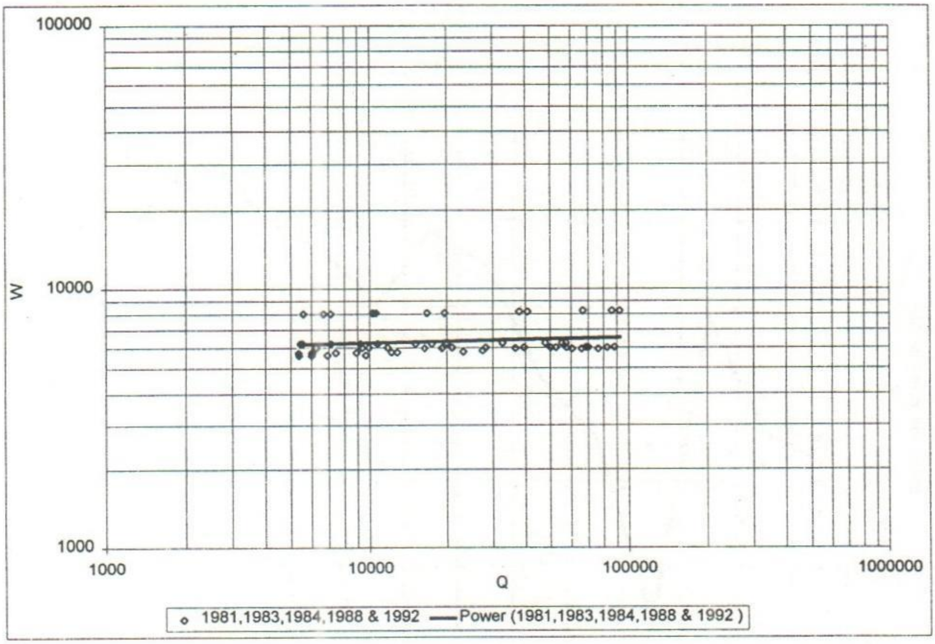


Figure 9 Variation of mean bed level for the standard BWDB cross-sections

Relationships of hydraulic geometric parameters with discharge

At Baruria Transit the relationships of hydraulic geometric parameters such as water width, water depth, flow velocity and water area versus discharge for the years 1981,1983, 1984, 1988 & 1992 are shown in **Figure 10** using monthly average data. From these plots it was observed that there exists a relationship between hydraulic geometric parameters.



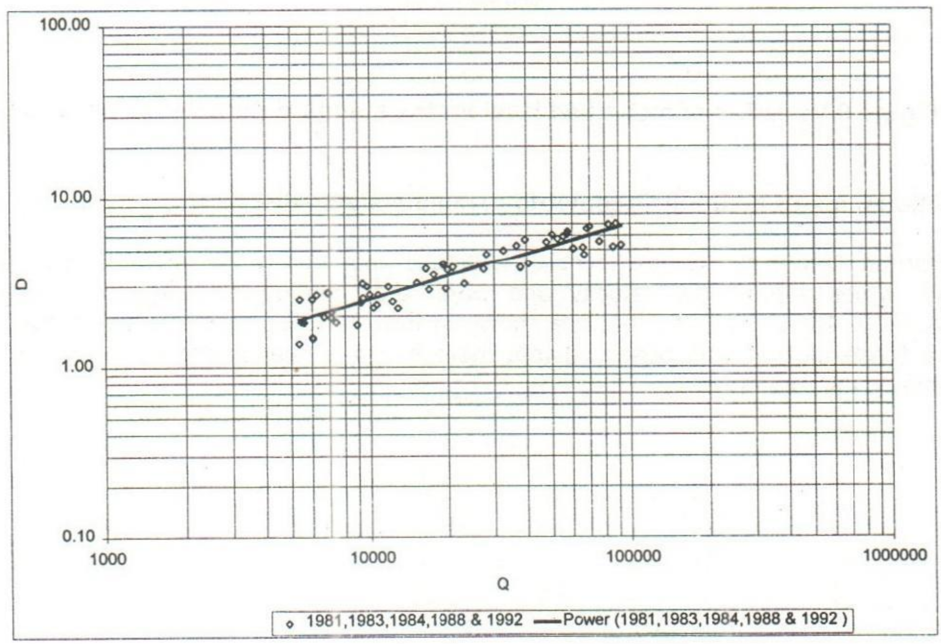
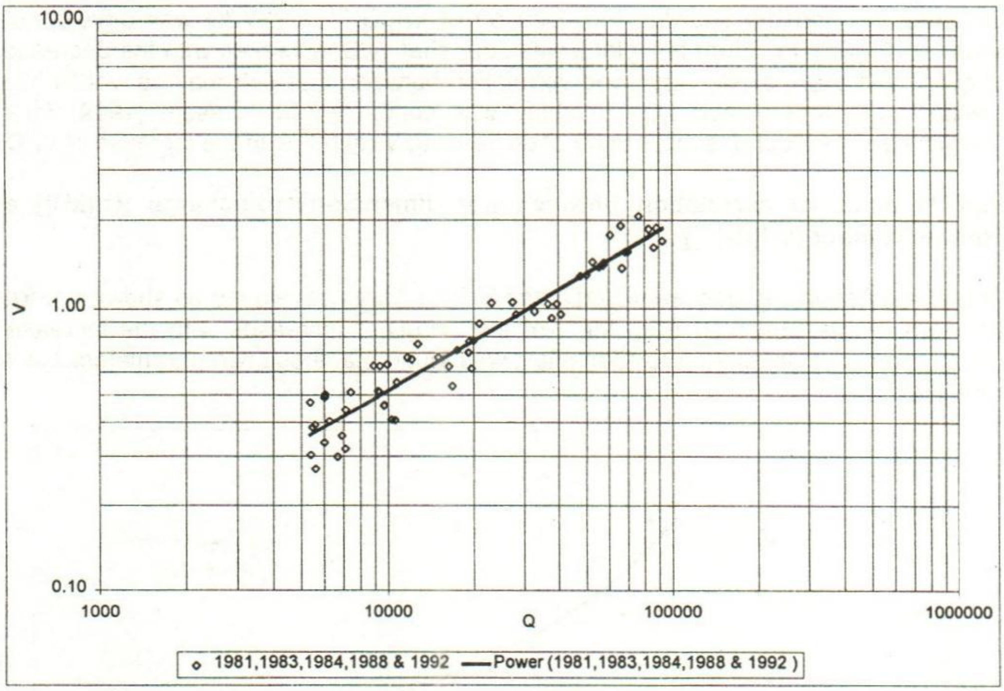
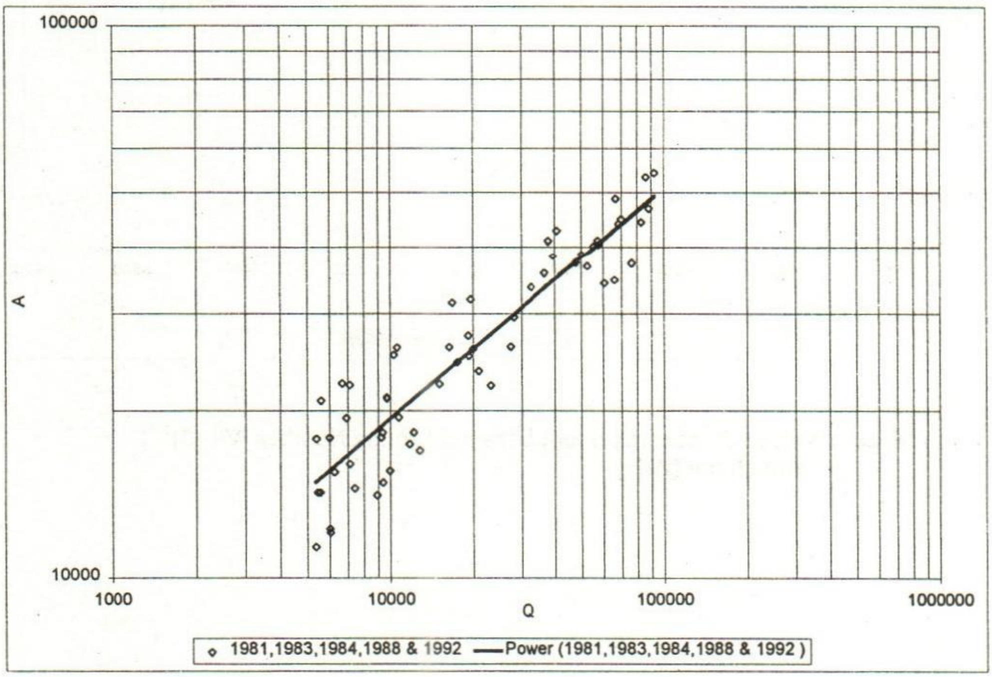


Figure 10 Relationships of hydraulic geometric parameters (W, D, V & A) with discharge (Q)

Figure 10 (Contd.)





Non-Dimensional correlation between Froude Number [$V/(gD)^{1/2}$] and Shape [W/D]

Graphical relationship showing the variation of $V/(gD)^{1/2}$ with W/D was developed as shown in **Figure 11**. From this plot it was seen that W/D increases with the decrease of $V/(gD)^{1/2}$ and there exists very good correlation between them. It may be worthwhile to mention here that similar type of study was conducted by Hossain (1989) for the Ganges river. He found similar trend of decreasing $V/(gD)^{1/2}$ with the increase of W/D.

Non-Dimensional correlation between non-dimensional discharge [Q/(kD)] and Froude Number [$V/(gD)^{1/2}$]

Graphical relationship between Q/(kD) and $V/(gD)^{1/2}$ was developed as shown in **Figure 12**. Here it was observed from the plot that $V/(gD)^{1/2}$ increases with the increase of Q/(kD). From the developed relationship it was found that these two parameters are well correlated.

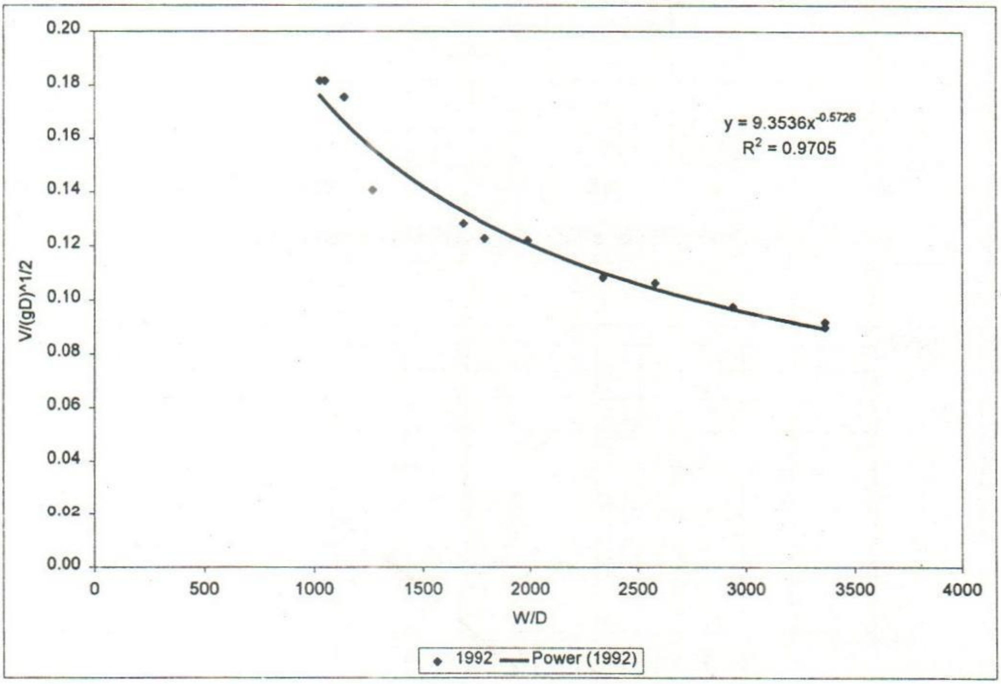


Figure 11 Non-Dimensional plot between Froude Number [$V/(gD)^{1/2}$] and shape [W/D]

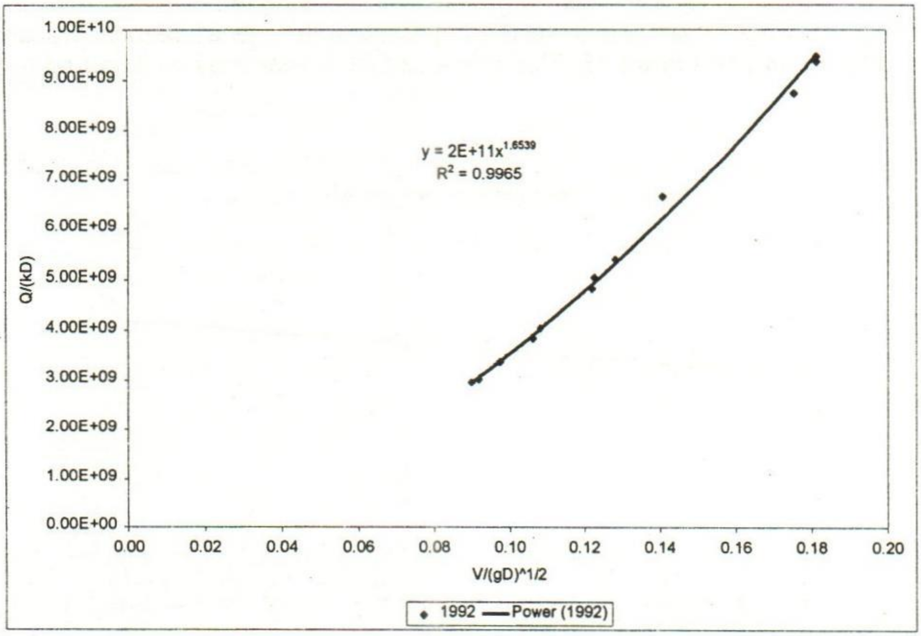


Figure 12 Non-Dimensional plot between non-dimensional discharge [Q/(kD)] and Froude Number $[V/(gD)^{1/2}]$

Variation of non-tidal water level and non-tidal discharge

A rating curve was developed at Baruria Transit from 1966 to 1994, which is shown in **Figure 13**. From this plot can be determined the water level with known discharge and vice versa.

Figure 13 Rating curve at Baruria Transit

At Baruria Transit from the separate plots of annual average discharge and water level against time, the annual average discharge had an increasing trend with time while the annual average water level had a decreasing trend with time at Baruria Transit as shown in **Figure 14** and **Figure 15**. This indicates bed erosion may be occurred at this station.

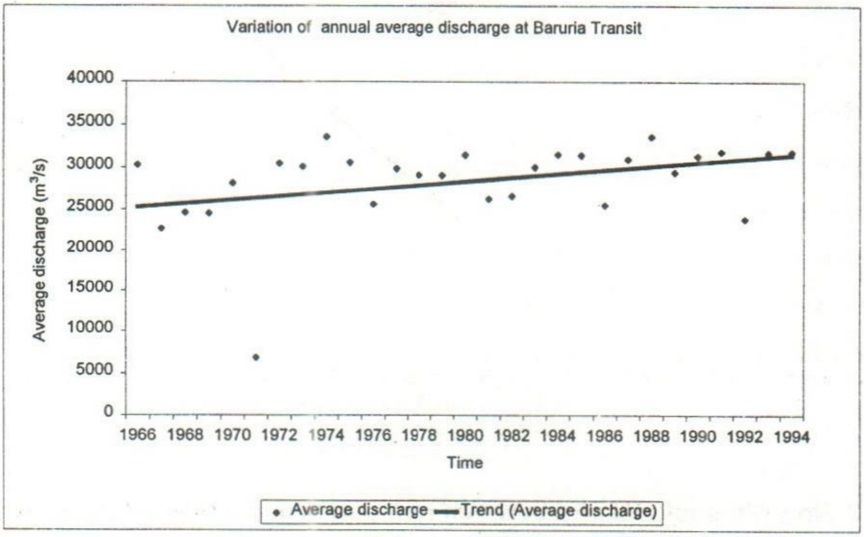


Figure 14 Variation of annual average discharge at Baruria Transit

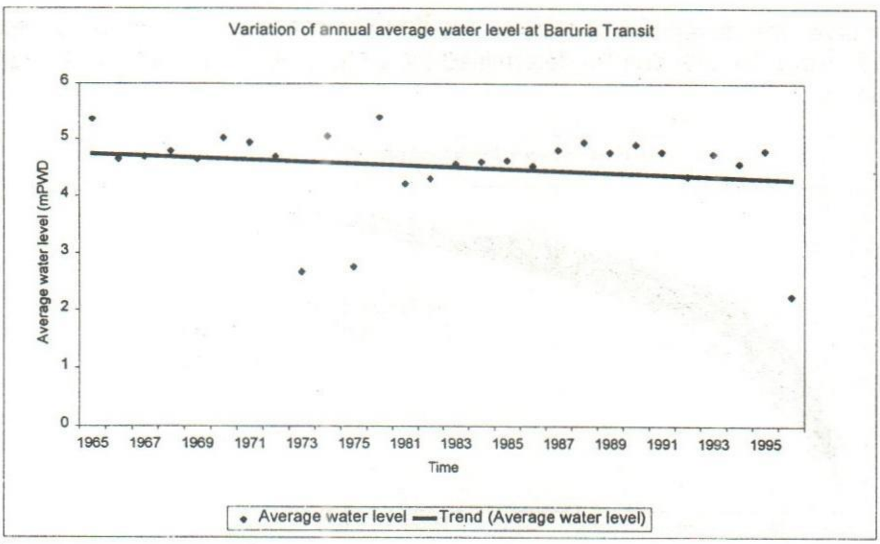


Figure 15 Variation of annual average water level at Baruria Transit

At Goalanda Transit from the plot of annual average water level versus time it was observed that the annual average water level had a slightly decreasing trend with time as shown in **Figure 16.** This also indicates erosion may be occurred at this station..

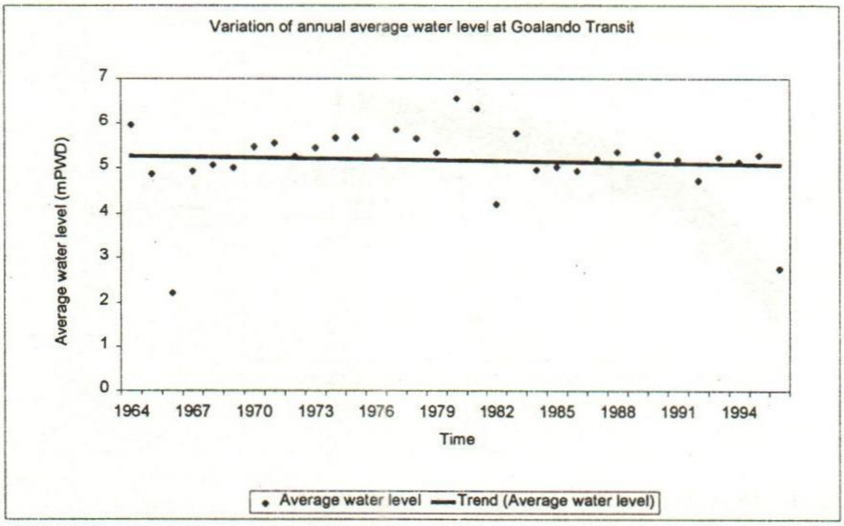


Figure 16 Variation of annual average water level at Goalanda Transit

Variation of tidal water level and tidal discharge

At Bhagyakul from the plot of annual average tidal water level (average value taken during ebb tide & neap tide) versus time it was observed that annual average tidal water level had a decreasing trend with time as shown in **Figure 17** indicating erosion may be occurred at this station.

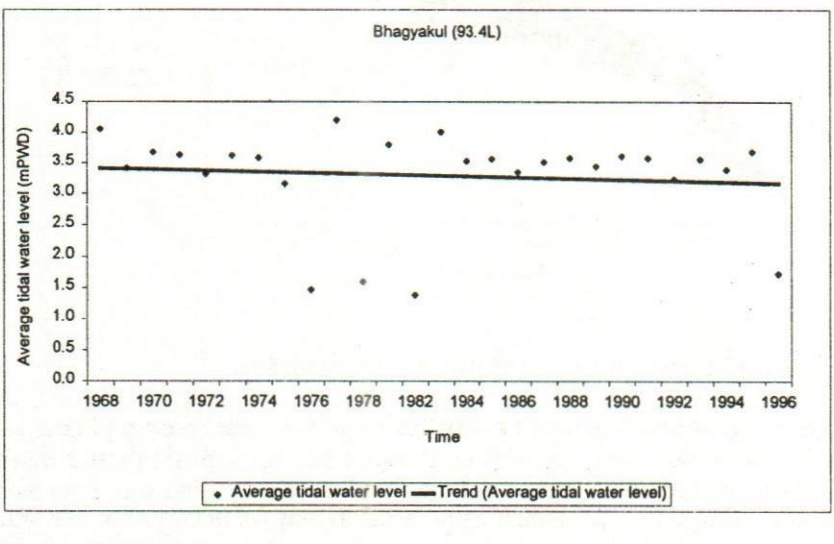


Figure 17 Variation of annual average water level at Bhagyakul

A rating curve was also developed at Mawa from 1968 to 1994, which is shown in **Figure 18** and **Figure 19** for ebb tide and neap tide respectively.

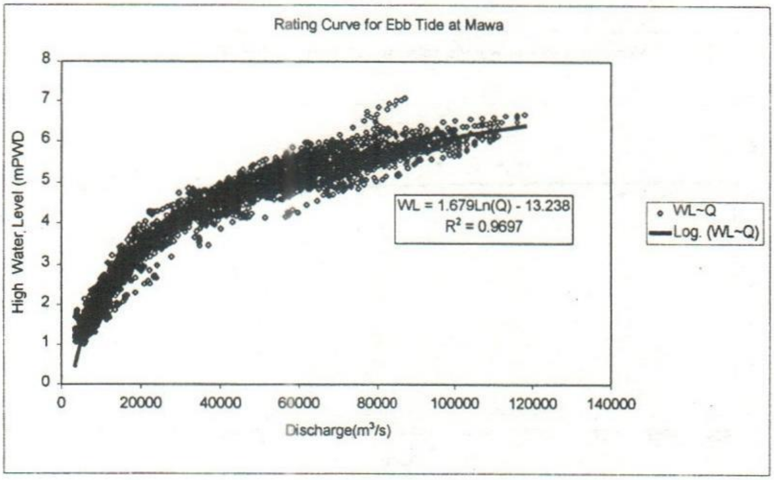


Figure 18 Rating curve at Mawa during ebb tide

From these plots one can determine the high water level and low water level with known discharge during ebb tide & neap tide respectively and vice versa at Mawa.

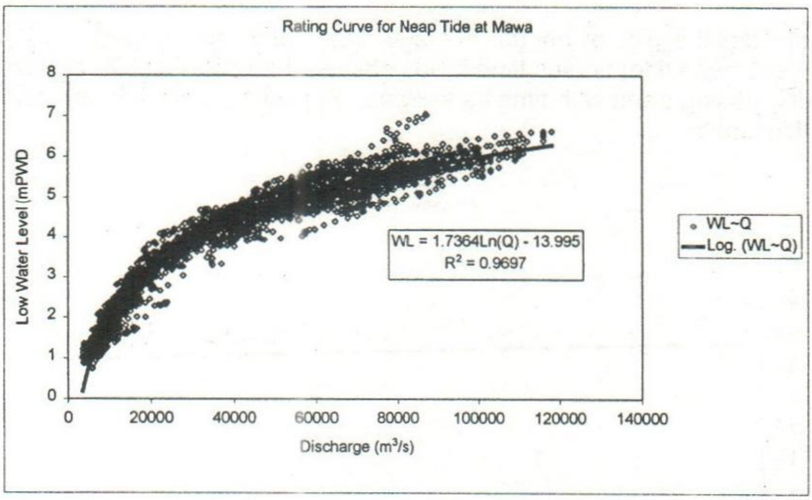


Figure 19 Rating curve at Mawa during neap tide

At Mawa from the separate plots of annual average tidal discharge and tidal water level against time, the annual average tidal discharge had an increasing trend with time and also the annual average tidal water level had an increasing trend with time as shown in **Figure 20** and **Figure 21**. This indicates no erosion may be occurred at this station.

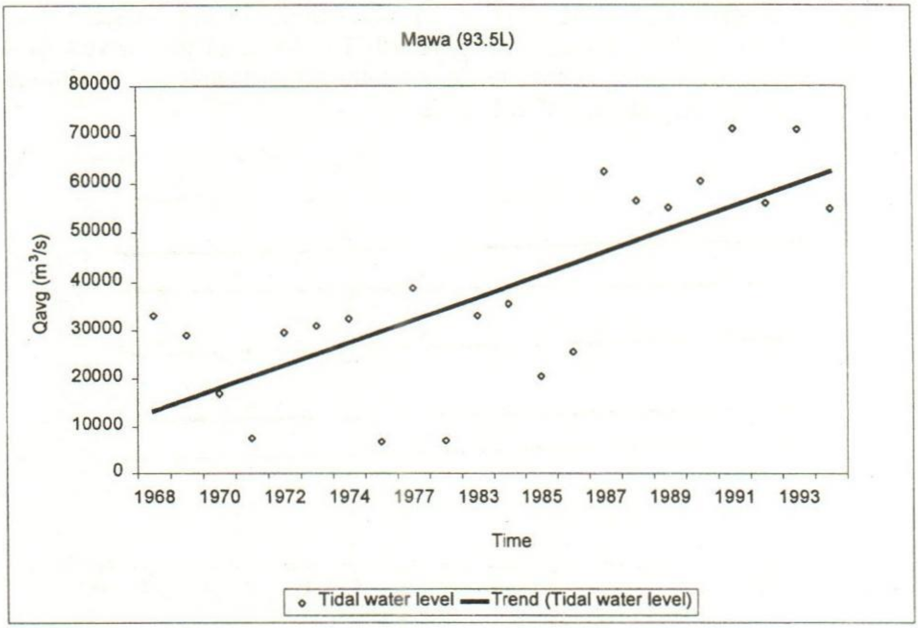


Figure 20 Variation of annual average discharge at Mawa

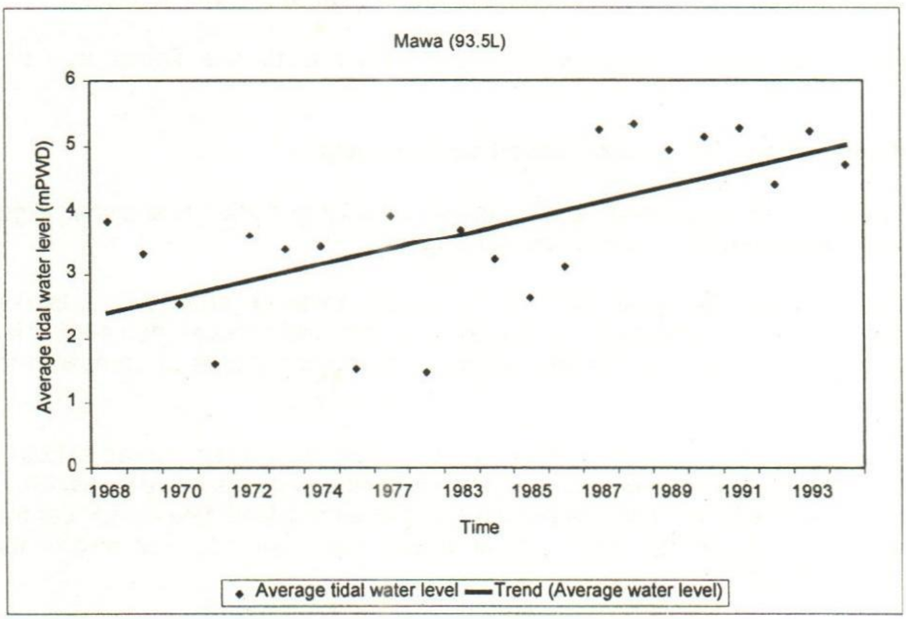


Figure 21 Variation of annual average water level at Mawa

Specific gauge analysis

Specific gauge analysis was conducted at Baruria Transit for the selected discharge levels from 1966 to 1993 as shown in **Figure 22**. The trend of the curves show that stages are slightly decreasing with time in sinuous pattern which characterizes the general dynamism existing at alluvial river beds.

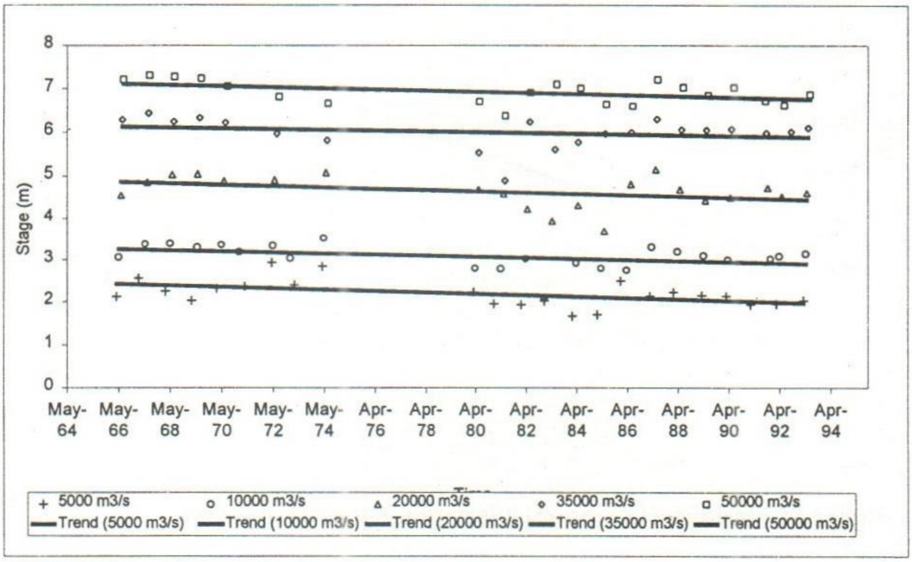


Figure 22 Variation of stages of Padma river at Baruria Transit

The figure also gives an indication that degradation at Baruria Transit may be taken place

Bankline shifting from superimposed satellite images

The superimposed map using satellite images of 1973 and 2001 is shown in **Figure 23**. The bankline movement is presented in **Table 2.**

It can be observed that in section-1 that the erosion was occurred at both banks. The left bank erosion was occurred about 1.7km and right bank was eroded 400m. It shows that during these years the left bank erosion was not significant as compared to other sections.

In section-2, it can be observed that in this section erosion was occurred at both banks. There was severe erosion observed in the left bank and its magnitude was 18km. The left channel formed a bend in this section and a large charland developed near the bend and still eroding seriously here. Serious erosion had been occurred also at the right bank and its magnitude was about 3km.

In section-3, the erosion and deposition was occurred in this section simultaneously. The right bank was eroded about 2.15km and the left bank deposited 1.4km.

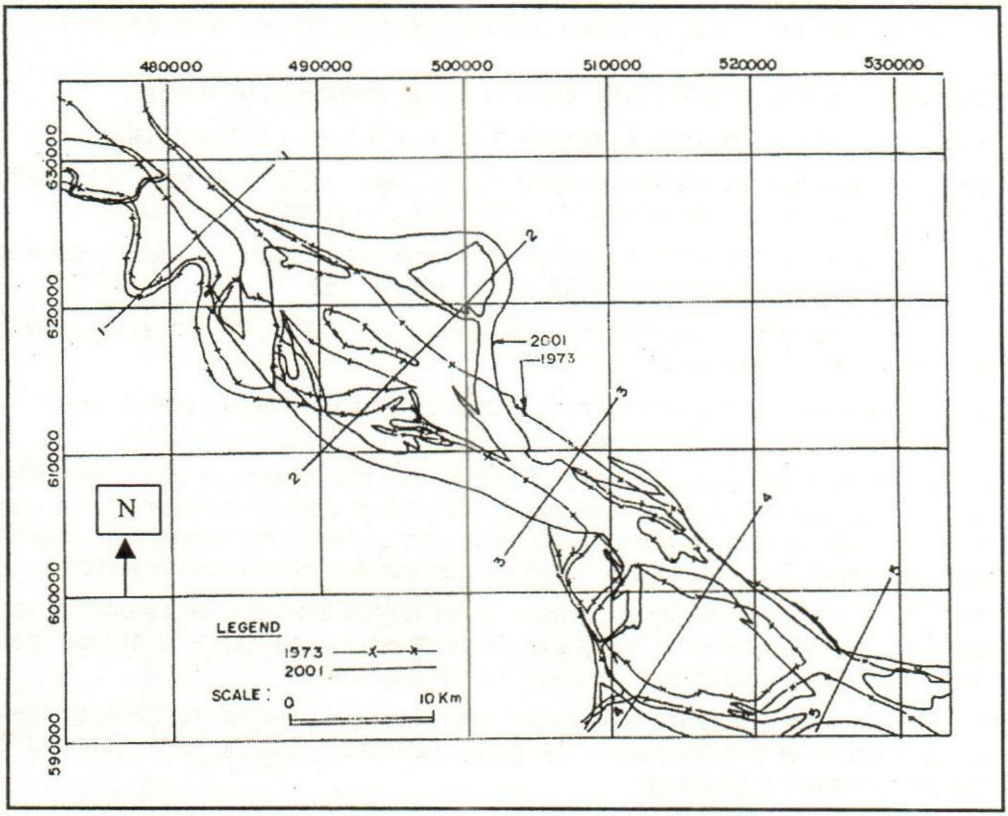


Figure 23 Superimposed map from satellite images of 1973 and 2001

In section-4 & 5, the erosion was occurred in the right banks and the left bank was stable i.e. no erosion and deposition was observed. The right bank was eroded about 900m in section-4 & in section-5 the serious erosion had been occurred in the right bank and its magnitude was 3.5km.

Table 2 Bank shifting along section lines of the Padma river from satellite images (1973-2001)

Section No.	Right Bank (km)	Left Bank (km)
1	-1.700	-0.400
2	-3.000	-18.000
3	-2.150	+1.400
4	-0.900	0.000
5	-3.500	0.000

NB: (+) means sedimentation and (-) means erosion

Conclusions

The following conclusions may be drawn on the basis of the present research study:

- Over a period of 1992 to 2000, the cross-sectional area varies randomly.
- The thalweg of the river moves from one bank to another in a random fashion.
- The mean bed level at maximum cross-sections rises in 2000 with respect to 1992, which represents the channel bed has a depositing tendency.
- A relationship is established between hydraulic geometric parameters against discharge for the years of 1981, 1983, 1984, 1988 & 1992.
- It is From the plot of non-dimensional parameters $V/(gD)^{1/2}$ versus W/D, W/D increases with the decrease of $V/(gD)^{1/2}$
- It is seen from the plot of non-dimensional parameters Q/(kD) versus $V/(gD)^{1/2}$, $V/(gD)^{1/2}$ increases with the increase of Q/(kD.
- It is seen from the separate plots of annual average discharge and water level versus time, it is found the annual average discharge has an increasing trend with time while the annual average water level has a decreasing trend with time at Baruria Transit. This indicates river bed erosion may be occurred at this station.
- From the plot of annual average water level versus time it is observed that the annual average water level has a slightly decreasing trend with time at Goalanda Transit indicating erosion may be occurred at this station.
- A rating curve is developed at Baruria Transit during the period from1966 to 1994 using water level and discharge. The general equation of rating curve obtained in the present study is given by:

  $$WL = 2.2281\ln(Q) - 17.221 \quad (1)$$

- From the plot of annual average tidal water level (average value taken during ebb tide & neap tide) versus time it is found that water level has a decreasing trend with time at Bhagyakul. This indicates erosion may be occurred at this station.
- From the plot of annual average tidal water level (average value taken during ebb tide & neap tide) and tidal discharge versus time it is found that both the water level and discharge have an increasing trend with time at Mawa. This indicates no erosion may be occurred at this station.
- A rating curve is developed at Mawa for the years from 1968 to 1994 for ebb tide and neap tide. The general equation of rating curve obtained in the present study is given by:

  $$WL = 1.679\ln(Q) - 13.238 \text{ (for ebb tide)} \quad (2)$$

  $$WL = 1.7364\ln(Q) - 13.995 \text{ (for neap tide)} \quad (3)$$

- At Baruria Transit specific gauge analysis for the selected discharge levels indicates that stages are decreasing in sinuous pattern. This means that scouring of river bed at this station may be occurred.

- From superimposed map from satellite images of 1973 & 2001, it is observed that various amount of shifting has been occurred at selected places.
- Maximum right bank erosion is 3.5km along section-5 and maximum left bank erosion is 18km along section-2 in 2001 as compared to 1973. No right bank erosion at the selected sections takes place and maximum left bank deposition is 1.4km in 2001 along *section-3 as compared to 1973.*

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Modeling of Salinity Intrusion process of Sundarban River Systems, Bangladesh

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Abstract

A simplified model for calculating salt concentration of river system has been developed for entire South West Region of Bangladesh including Sundarban area. The model result shows a reasonable agreement with observed data. A simplified model for seepage rate calculation has also been developed for the Sundarban river systems.

Introduction

The coastline of Bangladesh is about 710 km long with several tiny islands. Both natural and planted mangroves are available in the coastal area of Bangladesh, which lies between 21-23^0 N and 89-93^0 E. The natural mangrove forest includes the Sundarbans and the Chokoria Sundarbans. The Sundarban is the largest continuous mangrove forest in the world with an area 10000 km^2 of which 62% falls within the territory of Bangladesh (.A.F.M. Akhtaruzzaman, 2000).

As part of a naturally existing ecosystem mangroves forest play an important role in the preservation and protection of coastal and estuarine regions. Moreover with wise management the rich resources that the mangrove ecosystem offers could be of great assistance to the development of livelihoods for local communities. Nowadays mangrove ecosystem are undergoing widespread degradation due to a combination of physical, biological, anthropogenic and soil factors. A verity of human-induced stress such as change in water quality, soil salinity and sedimentation due to insufficient fresh water inflow from the upstream, conversion of mangrove wetlands for aquaculture, salt pans, and other land use are largely responsible for reduction of mangrove vegetation in Sundarban area.

Salinity is one of the most important determinants of mangrove forest growth and distribution. Mangroves develop well in places where salt concentration is between 20 and 35%. Too high salt concentration diminishes the number of species and their size. Various study also show that salinity influences physiological parameters, growth, survival and regeneration of mangrove plants. A general decline of the physiognomic structure of the forest is due to restricted growth of the trees, which is related to salinity is soil and river water (ESIAGRRP 2000). Moreover mangroves do also need a certain amount of freshwater during their growth. Fresh from rivers, channels, and rain dilutes the salinity of seawater, creating brackish water suited to many species during specific stages of their growth. In the present study an attempt has been made to develop a simplified one-dimensional mathematical model for understanding the salinity intrusion processes of the river system of South West Region including Sundarban river systems.

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Existing Salinity Condition of the Study Area

The salinity condition in the Sundarban River systems is highly depended on the salinity at the coast and volume of freshwater flow discharging from upstream (EIAGRRP, 2001). The South West region (Sundarban area) river system begins to affect by the salinity of the coastal water from the month of November. Salinity increases steadily from December through February, reaching maximum in late March and early April following the trend of the Ganges flow, where minimum flow in the Ganges appearing late April to May (SBCP, 2001).

During the monsoon, the large volume of freshwater discharging into the Bay of Bengal through the Meghna estuary leading to decrease salinity condition along the entire Bangladesh coastline (see Figure 1). The effect is reduced with the distance from the Meghna mouth and therefore the western part experiences much higher salinity levels compared to that in the eastern side. In the dry season, the freshwater flow decreases and consequently the salinity along the coast increases. The system, however never reaches equilibrium (SAWRMP, 1993)

Objective of the present study

Up to date no proper methodology has been developed for describing salinity intrusion process of the river systems of South West Region. In the present study the main objective was to develop an one dimensional mathematical model for salinity intrusion process of this area, which could be used in future for predicting salinity condition of the river systems due to any alternation took place in the river system of this area.

Data Collection

Usually salinity concentration of Sundarban river systems increases during dry season. To investigate the existing condition of the salinity intrusion process of the Sundarban river system during dry period, the data collection campaign took place at six locations in different river systems. The collected data were discharge, water level and salinity. The water level and discharge were collected for one day time series of 30 minutes interval, but salinity concentration has been collected only during slack time of high and low tide on that day.

Discussions

Collected water levels, discharge and salinity concentration have been analyzed for investigating the existing hydraulic condition and salinity concentration of different river systems.

Investigation of discharge and water level

From the analysis (Figure 3), it has been observed that peak flood discharge is significantly higher than peak ebb discharge for almost all of the river systems during the dry period. This might be happened due to seepage (absorption of water by soil) and shortage of freshwater inflow into the river systems.

Figure 1 clearly shows that only the Gorai River is the main source of fresh water inflow into the study area. But, nowadays it is observed that Gorai River flow has been drastically reduced during the dry period, which might be the cause for insufficient freshwater inflow in the study area.

Water levels inside the study area are highly dependent on the tidal water levels of the coast and to a lesser extent on the magnitude of the freshwater inflow from the upstream. Tides in the Bay of Bengal are semi-diurnal, exhibiting two high waters and two low waters per day. More over the daily, fortnightly and seasonal variations in water levels and tidal amplitudes experienced at the coast are also propagated inland during each tidal cycle. As the tidal waves travel up to the estuaries, they become distorted due to shoaling effects. The wave shape changes and amplification or dampening of the wave may occur. Table1 shows the variation of tidal range at several locations in side the study area and in the Figure 3 show the water profile of different river systems.

Table 1: Variation of tidal range at different locations

River	Location	Date	Tidal range (m)
Putia Khal	Harbaria	10.02.01	3.97
Selagang	Harintana	18.03.01	1.25
Pussur River	Digraj	13.01.01	3.19
Mrigomari	Mrigomari	17.02.01	1.69
Supati Khal	Supati	11.02.01	2.52
Sibsa River	Nalianala	27.02.01	3.63

Analysis indicated that during dry season the tidal range of different river systems (see Table 1) varies from 1.25 m to 3.97 m. The highest tidal range has been found in Putia khal at Harbaria location is 3.97 m.

Seepage Model

Seepage also plays an important role for decreasing tidal volume in the river system of Sundarban area during dry season. To investigate the seepage rate of different river systems in Sundarban area, an equation (8) has been developed for calculating seepage rate of the different river systems of this area considering: (1) water level is horizontal, (2) water level changes sinusoidal ($\eta_{(t)} = -(H/2)*\cos\omega t$), (3) wall of a basin is vertical.

The development of seepage model has been expressed elaborately through Figure 2 ,

$Q = Q_{amp} * \sin \omega t + qA_{T},$ .... (1)

$Q_{amp} = (A_T \pi H)/T$...................................... (2)

Where: Q = discharge (m^3/s); A_T = area of assumed tidal basin (m^2); H = tidal range (m) ; T = tidal period ; t = time ; $\omega = 2\pi/T$; q = seepage rate per unit area,

$Q_{max} = Q_{amp} + qA_{T},$.. *(3)*

$Q_{min} = - Q_{amp} + qA_T$,....................................(4)

$Q_{max} + Q_{min} = 2qA_T$,....................................(5)

$Q_{max} - Q_{min} = 2Q_{amp}$,....................................(6)

now,

$(Q_{max} + Q_{min}) / (Q_{max} - Q_{min}) = q^* A_T / Q_{amp}$,..........(7)

$q = [(Q_{max}+Q_{min})/(Q_{max}-Q_{min})] * \pi H/T$,................(8)

$\{Q_{max} \neq Q_{min}\}$.

The calculated seepage rate using equation (8) is given in **Table 2.**

Table 2: Seepage rates of different river systems

River name	Location	Q_{max} (m^3/s)	Q_{min} (m^3/s)	T (sec)	Tidal range H (m)	q (m/s)
Sibsa	Nalianala	16637	-20399	43200	3.63	-2.68×10^{-05}
Putia Khal	Harbaria	63	-133	43200	3.97	-1.3×10^{-05}
Mrigomari	Mrigomari	956	-1145	43200	1.69	-1.10×10^{-05}
Pussur	Digraj	2088	-2524	43200	3.19	-2.19×10^{-05}
Selagang	harintana	2214	-2554	43200	1.25	-6.48×10^{-06}
Supati Khal	Supati	1979	-1973	43200	2.52	2.78×10^{-07}

From the **Table 2** it is seen that seepage rate of all the river systems is more or less same, there is a negligible variation among themselves. It may happened due to same bed material of all the river systems. The bed material of all the river systems has been found fine sand. A gross comparison have been made between calculated seepage rate and the value of coefficient of permeability for fine sand ($k = 10^{-5}$ m/s) and found more or less satisfactory result. From the estimated seepage of different rivers it may be concluded that seepage rate of all the river systems in Sundarban area is almost same and very small in amount.

Analysis of salinity concentration

Analysis of secondary and primary data indicates that salinity concentration of Sundarban river systems fluctuates semidiurnal in a period of about 12.5 hours due to flood and ebb tides. It varies fortnightly due to the spring and neap tide. It has an annual cycle due to variation of upland discharges. There are also long-term trends or cycles due to astronomical and meteorological changes such as climatic changes, which are expected to produce higher sea levels. From the Table 3 it has been seen that variation of salinity concentration during high and low tide in the river systems of Sundarban area is negligible. Maximum salinity concentration has been found in Nalianala at Sibsa river is 9.31 ppt and minimum salinity concentration found in Digraj at Pussur River is 1.56 ppt.

Table 3: Salinity concentration of different river system in Sundarban area

River	Location	Salinity (ppt)	
		Slack, high tide	Slack, low tide
Putia Khal	Harbaria	6.38	5.25
Selagang	Harintana	8.25	7.01
Pussur River	Digraj	1.53	1.56
Mrigomari	Mrigomari	6.37	6.39
Supati Khal	Supati	5.00	4.92
Sibsa River	Nalianala	8.36	9.31

Salinity Model Development

Salinity intrusion is predominantly not a problem only for Sundarban river systems, but it is also a problem for entire South West Region river systems. Due to shortage of freshwater inflow a substantial amount of salt concentration has been increased in the river systems of South West Region during dry season. For over all understanding and quantifying the behavior of the salinity intrusion process of river systems, a simplified one dimensional mathematical model has been developed for the river system of this area.

Governing Equation

Mathematically the problem of salinity intrusion in an estuarial network may be defined by the mass conservation equation for salt in the river system. For one-dimensional flow this equation is as follows,

$$\partial A_cC/\partial t = \partial/\partial x(KA_c\partial C/\partial x) - \partial QC/\partial x - qBC \ldots\ldots\ldots\ldots(9)$$

Where: A_c = cross sectional area, K = diffusion coefficient, C = concentration of salt, Q = discharge, q = seepage rate per unit area, B = width of the river.

The above-mentioned equation (9) has been developed considering the following assumptions and conditions:

river width $B(x)$ is larger than depth $h(x)$ [$B(x) >> h(x)$], Where 'x' is a distance taken along the river length,

- water surface is a function of x and t, where t is time [$\eta = \eta(x,t)$],
- consider the convective transport,
- consider seepage,
- consider diffusion by Fick's law.

Equation for average salt concentration

For assessing the salinity condition of a river system, it is always appreciable to know the average salt concentration of the river system. To calculate average salt concentration of a river system, a simplified equation (10) has been derived from the governing equation (9) considering linear assumption. The equation is as follows:

$$C_{avg} = C_1 * e^{\sqrt{(q/Kh)} * x} + C_2 * e^{-\sqrt{(q/Kh)} * x}, \ldots\ldots \quad (10)$$

$$C_1 = [C_L - C_w * e^{-\sqrt{(q/Kh)} * L}] / [2\sinh(\sqrt{(q/Kh)} * L)], \ldots \quad (11)$$

$$C_2 = C_w - C_1, \ldots\ldots \quad (12)$$

$$U_* = [\{ n^* (g)^{1/2} \} / R^{1/6}] * V, \ldots\ldots \quad (13)$$

$$K = a^* U_* * h, \ldots\ldots \quad (14)$$

where: C_{avg} = average salt concentration of the river system,
C_1 & C_2 = constants,
q = seepage rate per unit area (m/s),
K = diffusion coefficient,
h = average depth of river (m),
x = distance from the bay (m),
L = Total distance of the river system from the bay (m),
C_w = Average salt concentration of the bay (ppt),
C_L = Salt concentration at the end of the river system from the bay (ppt),
V = average velocity (m/s),
U_* = friction velocity (m/s),
R = hydraulic radius (m) ; $R = h$ (in case of wide channel),
a = adjustment coefficient,
n = roughness coefficient.

Model Application

The Pussur-Rupsha-Nabaganga-Gorai is the major river system of South west Region (see Figure 1). This river system has both tidal and non tidal character. The tide goes up to the station Bardia on Nabaganga River. To understand the salinity intrusion process of this river system, the equation (10) has been applied for calculating average salt concentration at Mongla on Pussur river and at Khulna on Rupsha river considering well mixed salt concentration in the river system. Initial condition of the salt concentration has been fixed up in the Bay near at Hiron Point on Pussur river is C_w = 23 (ppt) and at end of the river systems near Bardia on Nabaganga river is C_L = 0 (ppt), where salt concentration is almost zero. Distance from the Bay to end of the river system is L= 175000 (m). Estimated average seepage rate for the river system of South West Region is taken q = 10^{-10} (m/s), average depth of the river systems is h = 5 (m) and average velocity in the rivers system is taken as v = 0.6 m/s, roughness coefficient n = 0.033. Figure 4 shows that model result fairly matched with average measured salt concentration. Here it is necessary to mention that influence of other rivers in the confluence has been ignored in the calculation.

Summary and Conclusion

Increase of salinity concentration in the river systems of Sundarban area due to decrease of freshwater inflow during dry season is one of the main reason for sedimentation of river systems. Seepage of tidal discharge is one of the reasons for increasing soil salinity in this area. A qualitative assessment for salinity condition of the river systems of this area is necessary for taking any measurement for healthy hydraulic condition of the river systems of this area. Mathematical model for salinity intrusion process of this area could be a appropriate tool for assessing the salinity condition of this area.

A simplified model for calculating salt concentration of the river systems of South West Region including Sundarban area has been developed. The main conclusions of the study are as follows,

- Flood tide is higher than ebb tide for all the river systems, due to seepage and shortage of fresh water inflow into the river systems.
- Seepage rate for all the river systems of Sundarban area is more or less same due to same bed materials of the river systems.
- Model results show a reasonable agreement with measured salinity data.

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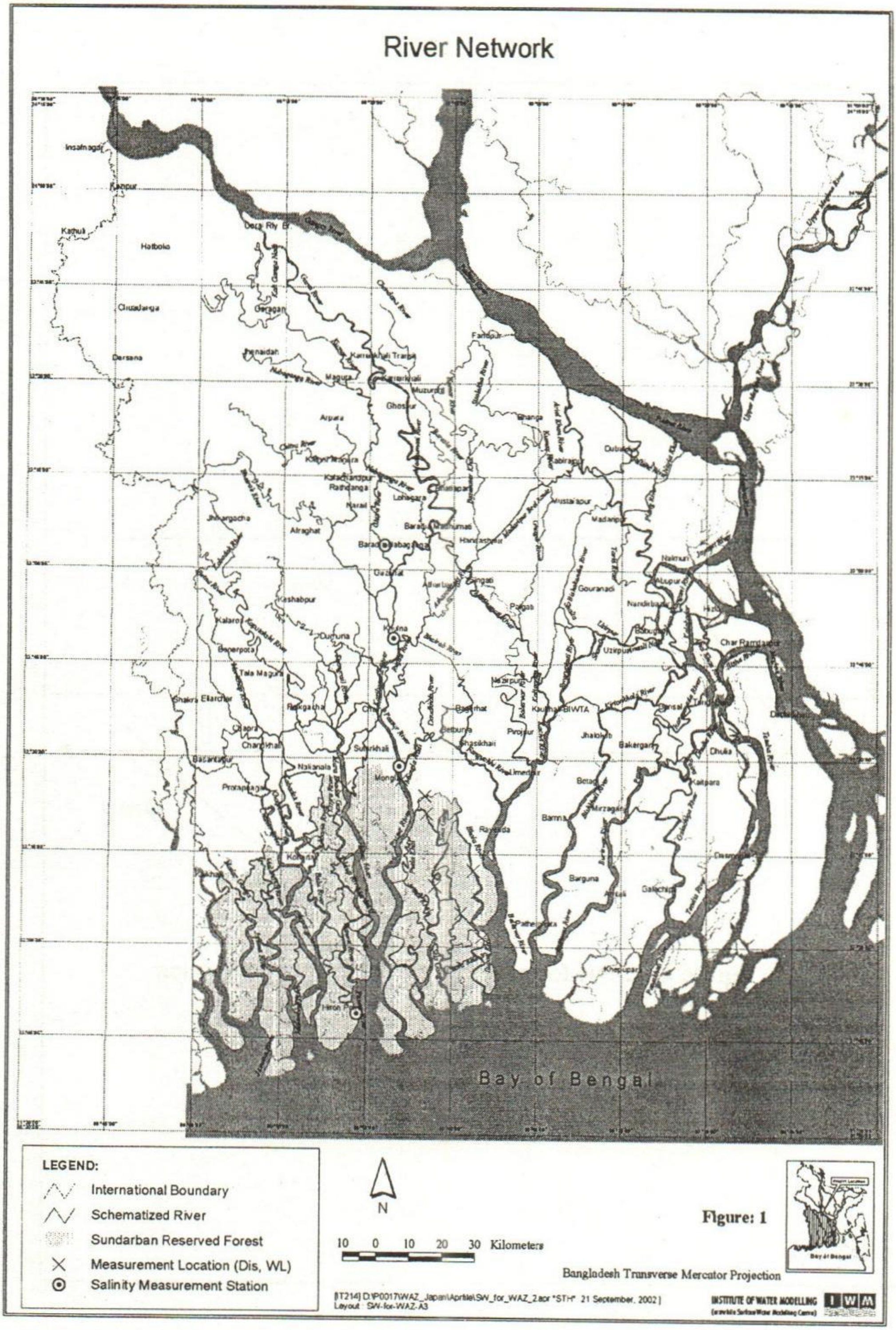
River Network
Bay of Bengal
LEGEND:
International Boundary
Schematized River
Sundarban Reserved Forest
Measurement Location (Dis, WL)
Salinity Measurement Station
N
10 0 10 20 30 Kilometers
Bangladesh Transverse Mercator Projection
INSTITUTE OF WATER MODELLING

Figure: 1

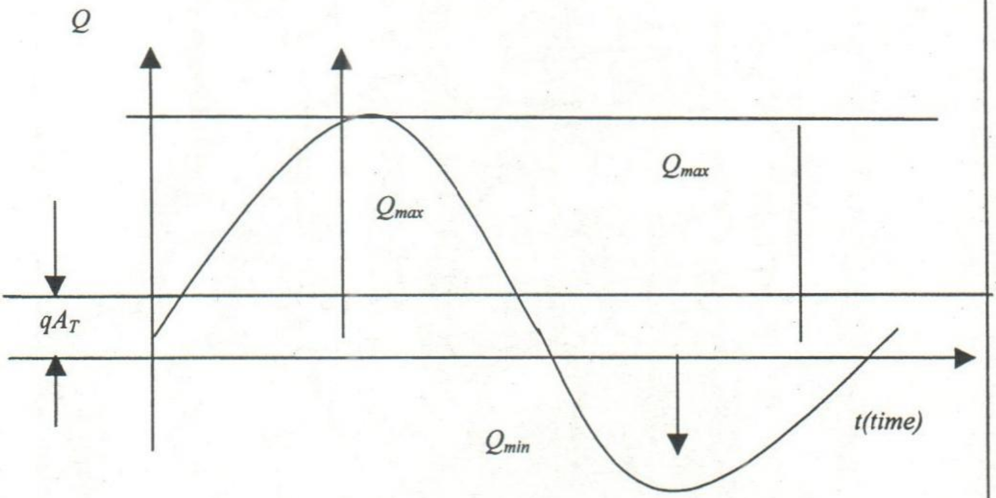

Figure 2: Basis for development of seepage equation

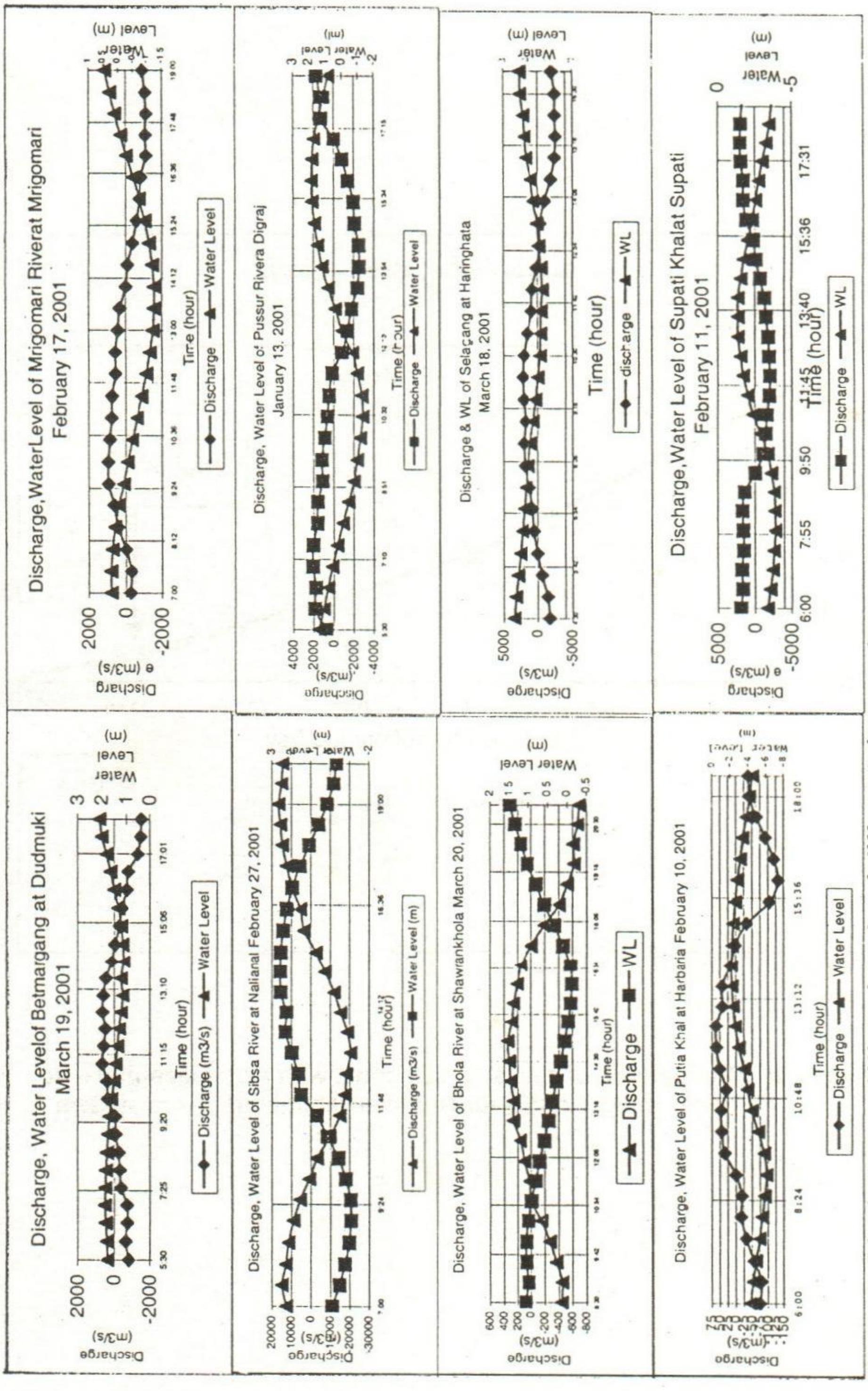

Figure 3: Discharge and Water Levels of different river systems

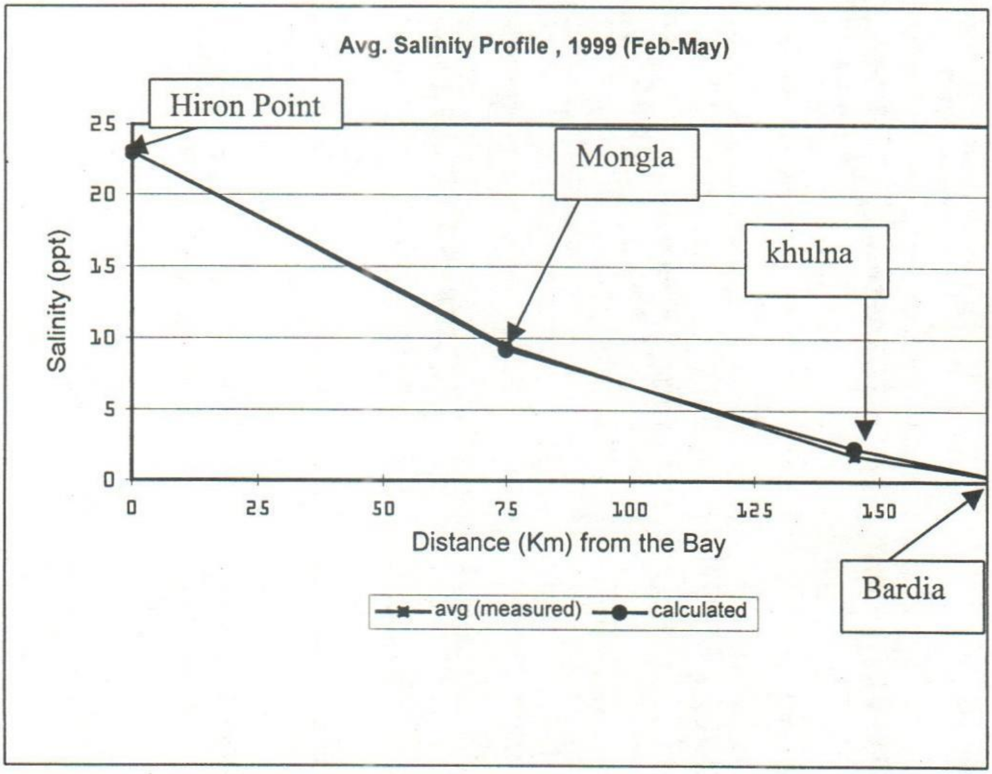

Figure 4: Comparison of calc.avg. salt conc. with avg. measured salt conc. along the Pussur-Rupsha-Nabaganga rivers system

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ROLE OF SEDIMENT LOAD IN DETERMINING HYDRO-MORPHOLOGICAL PARAMETERS: A STUDY FOR THE RIVERS GANGES, GORAI AND MATHABHANGA

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Abstract

The rivers of Bangladesh have been undergoing morphological imbalances causing large siltation in the riverbeds. The drainage area of the two major rivers namely the Ganges and the Brahmaputra amount to about 1111,000 km^2 and 550,000 km^2 respectively. During monsoon huge amounts of sediment enter these rivers from their vast drainage basins together with water. The combined transport of water and sediment is a three-dimensional time depending phenomenon, which is of a complex nature. The occurrence of either an extremely high discharge or a sediment discharge can have large influence on the fluvial process. This is due to the strongly non-linear relationship between water movement and sediment movement. In the present study the hydrological, hydraulic and sediment data of three rivers for a given time period have been analyzed and plotted to see the influence of variation in the sediment load on hydro-morphological parameters of the rivers. The sediment transport rates have been determined from the suspended sediment samples and no information is available about bed load transport rates. However, the outcomes of the study can still be useful to recognize the role of the sediment load in determining hydro-morphological parameters.

Introduction

Bangladesh is a flat deltaic country. The sediment deposits of the three great rivers namely the Brahmaputra-Jamuna, Ganges-Padma and the Meghna and numerous other rivers formed the main land of this country since time immemorial. It has a unique system of rivers that plays an important role in the development of overall economy of the country. The rivers are getting silted up every year. The deposition of sediment in the river and drainage channel bed is creating serious problems for navigation, flood inundation, irrigation and maintenance of river ports and harbors. The flood has become almost a regular phenomenon in this country causing serious damage to standing crops, lives and properties. Due to deposition of sediment in the riverbed BIWTA is spending a large sum of money for dredging riverbeds to maintain navigable water depths.

The river courses are extremely unstable due to large variation in discharge and sediment load. The water environment and the socio-economic condition of the people of Bangladesh are significantly influenced by the characteristics of the rivers. The characteristics of the sediment load coming from the upstream and the characteristics of the local riverbed and bank material are playing important role in determining the river channel hydraulic parameters. The sediment transport rate is related to the size of the material. The coarser the bed material the smaller the transport rate. Even the sediment

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deposition depends on its size. The smaller the particle sizes the smaller the settling velocities and slower the particles settle. The bar deposits, the composition of bank material and flood plains, the channel geometry and planform of the rivers are influenced by the sediment characteristics. The study aims to develop understanding regarding likely responses of the rivers to a variation in the sediment load.

Description of the rivers

The Ganges

The Ganges is one of the largest rivers of Bangladesh. It is noted for its massive water discharge and huge amount of sediment load during monsoon and low flow during dry season. The Ganges River originates from the west of Nanda Devi range in Himachal Pradesh and northernmost Uttar Pradesh, west of Nepal. The Ganges basin includes the entire territory of Nepal and Uttar Pradesh of India. The total length of the river from its source to Daulatdia is about 2200 km and its drainage area is about 1.1 million sq. km. Among the three major rivers the Ganges has the highest drainage area yet its water yield is the least of the three rivers (Barua 1994). This is primarily due to the distribution of rainfall in the drainage basins that is higher in the eastern drainage basins and lower in the western drainage basins. The other notable characteristics of the Ganges are its wide meandering planform and average water surface slope of about $5x10^{-5}$ (Khan and Barua 1994). The bank line migration is very erratic in nature and varies widely from bend to bend as well as year to year. The bed material size is about 0.14-mm (D_{50}) (RSP Report 1996). The mean annual discharge is about 11000 m^3/s and the bankfull discharge is about 43000 m^3/s.

The Mathabhanga

The river Mathabhanga is one of the distributaries of the Ganges. It takes off from the Ganges at Jalangir in the district of Murshidabad of West Bengal. It enters Bangladesh at Insafnagar in the district of Kushtia. Then it flows towards the south through Daulatpur, Bheramara, Alamdanga, Chuadanga and Meherpur. It again enters Indian territory near Chuadanga and finally discharges into the Bhagirathy after joining the river Churna in West Bengal. Within Bangladesh territory its length is about 130 km and average width is about 190m (Wazed 1991). Kumar, Chitra, Naboganga and Kabodak are its main distributeries. The offtake position of the river is very unstable. The minimum and maximum water discharges recorded at Hatboalia from the year 1998 to 2000 are 4.5 m^3/s and 268 m^3/s respectively.

The Gorai

The river Gorai is an important distributary of the Ganges. It takes off from the right bank of the Ganges at Talbaria, 19 km downstream of Hardinge in the name Gorai. It flows through the district of Kustia and enters the district of Jhenaidah at Goneshpur. Therefrom it travels along the border of Kushtia-Jhenaidah and enters Rajbari district at Chadat. From Rajbari it flows along the border of Faridpur-Magura in the name Gorai-Modhumati to enter the district of Bagerhat. Then it flows through the district of Barishal and falls into the Bay of Bengal at Haringhata. Gorai is a long and wide river. The total length of the river from its source to Haringhata is about 372 km of which 89 km upto

Mohammadpur in the district of Magura is named as Gorai; therefrom upto 137 km downstream it is named as Madhumati and in the remaining 146 km upto Haringhata it is known as Baleshwar. Kumar. Kaligonga, Dakua, Burigorai etc. are its main distributeries and Chandana is the tributary of Gorai. The river Gorai is a very old river and its former name was Gouri. The water level gauging stations of the river Gorai are situated at GORAI RLY Bridge, Kamarkhali, Nazirpur and Pirozpur. As the river Gorai flows through the districts of Kushtia, Jessore, Faridpur, Khulna and Barishal the agricultural development and water environmental condition of these region are largely dependent on use of water of this river. Due to large-scale siltation at the Gorai mouth the offtake of the Gorai was completely disconnected from the Ganges particularly in the dry season. Due to reduced dry season flow of the Gorai the environmental quality of southwest region of Bangladesh is at stake. In 1999 Government of Bangladesh took up a project named Gorai River Restoration Project (GRRP) with financial assistance of donor agencies to augment the dry season flow through the Gorai. The maximum and minimum water discharges recorded at Gorai Railway Bridge during the year from 1998 to 2000 are 6145 m^3/s and 103 m^3/s respectively. On the other hand those values measured at Kamarkhali for the same time period are 4550 m^3/s and 57 m^3/s respectively.

Literature Review

Sediment transport mechanism and classification of sediment

When the flow condition reaches the critical stage that is when the value of bed shear velocity just exceeds the critical value for initiation of motion, the bed material particles start moving by rolling or sliding in continuous contact with the bed. With the increase of bed shear velocity the particle moves along bed by more or less regular jumps that are called saltation. When the value of bed shear velocity begins to exceed the fall velocity of particles, the sediment particles can be lifted to a level at which the upward turbulent force will be of higher order than the submerged weight of the particles, as a result the particles may go into suspension. At any time a good number of particles may jump but eventually some of them return to bed again; by that time other particles go into suspension and the process goes continuously. As a result depending on the magnitude of the turbulence, the shape and size of the bed particles some of them remain in suspension and move along with the flowing water. The main sources of sediment are materials of streambed, bank material due to erosion and the fine materials that come from the catchment of the river. According to sources sediment can be divided into two groups viz, bed material load and wash load but according to transport mechanism sediment can be classified into two categories viz, bed load and suspended load (Jansen, 1979). Suspended load consists of sediment particles held in suspension by balancing their gravitational force with upward forces due to turbulence of the fluid. The wash load is defined as material (finer than some 50 micron) than the bed material, which originates from upstream catchment area or from bank erosion. It is independent on capacity of the river and almost uniformly distributed throughout the river. Bed load is characterized by the movement of the bed material by rolling, sliding and small jumps. Bagnold defined (ASCE, 1977) the bed load transport as the sediment transport in which the successive contacts of particles with bed are strictly limited by the effect of gravity.

Causes of Sediment deposition

During monsoon huge amounts of sediment enters these rivers from their vast drainage basins together with water. The combined flow of water and sediment makes the river behavior very much complex. Since the country is a deltaic flat plain the rivers can not carry so much sediment and sediment deposition occurs on the riverbeds as well as on the flood plains eventually. In fact delta building process is still going on through deposition of sediment every year. It is visible in the southern seaward portion of the land. A substantial portion of suspended sediments brought down by the three mighty river systems remain close to the coast, floating in the water due to the turbulence caused by the waves. During high tide all the low lying lands are inundated and when the flow stops the sediments are deposited on the land.

Factors influencing the river morphological parameters

The discharge and sediment transports of a river vary with time and it has large influence on morphological parameters. The bed material transport is determined by the composition of the bed and by the hydraulic characteristics of the river. On the other hand the amount of wash load in a reach is only determined by the upstream supply. Hence it is not determined by the hydraulic parameters of the stream. The grain size, valley slope and human interference by major river training can influence the shape of the river. Moreover, the river may change its shape as a function of time.

Methodology

Data collection

A large number of suspended sediment and bed material samples of the Ganges, Gorai and Mathabhanga collected by BWDB have been tested in the sediment laboratory of RRI during the period from 1998 to 2000. The cross sectional data of the Ganges are collected from SWMC, Dhaka. The hydrographic chart of the Ganges and Gorai are collected from BIWTA. The cross-sectional data and hydrographic chart that are collected from SWMC and BIWTA are not sufficient, consistent and regular. The discharge and water level data of the Gorai, Ganges and Mathabhanga are collected from Bangladesh Water Development Board (BWDB). The data is collected for 2 (two) to 4 (four) years between the year 1997 and 2001. Detail discharge measurement data is collected for the purpose of the study.

Analysis of sediment data of the rivers

In order to study the sediment characteristics of the Ganges a large number of suspended and bed material samples collected by BWDB from the Ganges at Hardinge Bridge have been analyzed. The grain size distribution shows that the bed material consists mostly of sand (80% to 100%) with small fraction of silt particles (0% to 20%). The D_{50} varies from 0.08 mm to 0.20 mm (RRI Report Sed-75).The suspended sediment samples have been tested for determining sediment concentration and grain size distribution. The results of grain size distribution show that in the suspended load the silt percentage varies from 25% to 65% whereas the sand percentage varies from 35% to 75%. The D_{50} ranges between 0.08 mm to 0.11 mm. The suspended sediment transport rates have been calculated from concentration values and corresponding water discharges (Sed-7, 1999, Sed-4, 2001).

To study the sediment characteristics of the Gorai a large number of suspended sediment samples collected by BWDB from the river Gorai at Gorai Rly. Bridge and Kamarkhali during the year 1998 to 2000 are tested in the sediment technology laboratory of RRI. From the suspended sediment samples sediment concentration has been determined (Sed-2, 2001, Sed-4, 2000,Sed-8, 1999). From these concentration values and corresponding flow discharges suspended sediment transport rates have been calculated. A large number of suspended sediment samples collected by BWDB from the river Mathabhanga at Hatboalia during the period from 1998 to 2000 have also been tested in the sediment technology laboratory of RRI. From the tests suspended sediment concentrations have been determined (Annual Report, 1998, 1999 &2000). The suspended sediment transport rates are calculated from the suspended sediment concentrations and corresponding water discharges.

Analysis of hydrological and hydraulic data

The collected data is processed first. The processed data is then used to analyze the trend in temporal variation of suspended sediment transport with flow discharge. It is done for all rivers selected for the study. Then trend analysis is made to see the temporal variation of dependent hydraulic variables with flow discharge. In some cases temporal interrelationships among different dependent variables have also been established. River channel morphological parameters vary with flow and sediment transport. Here the analysis is made to see the temporal variation of dependent hydraulic variables with flow discharge. This analysis will enable the reader to visualize the temporal variation of the hydro-morphological parameters with flow discharge due to variation in the suspended sediment load. It is to be noted here that some other factors may also influence such variations.

Results and discussions

The Ganges

The water discharges and the corresponding suspended sediment transports recorded at Hardinge bridge have been plotted in Figure-1 for the years from 1998 to 2000. It can be seen from the figure that transport rates have decreased a bit in 1999 for discharge upto about 35000 m^3/s and substantially increased for discharge higher than that compared to the situation in 1998. In fact large sediment transport occurred in 1999. On the other hand sediment transport rates are seen to have decreased further in 2000 compared to the both 1998 and 1999 situation.The water levels and corresponding discharges recorded at Hardinge Bridge have been plotted for the years 1998 to 2000 and shown in Figure-2. From the figure it is noticeable that water levels have dropped slightly in 1999 in comparison with situation in 1998 particularly for higher discharges. However, in 2000 situation the water levels have again attained the almost 1998 condition.

In order to investigate the effects of variation in the suspended sediment transport on downstream hydro-morphological parameters two cross-sections are selected. One is at

Talbaria (G11), just upstream of Gorai offtake and another (G10) is at about 9 km downstream of the first one. The cross-sectional data is collected for two years namely 1998-99 and 1999-2000. The cross-sectional survey was conducted in the wintertime (December-January). From these cross-sectional data it is possible to investigate the influence of large increase in the suspended sediment transport on hydro-morphological parameters of the river. For this purpose trend analysis is made to see the variation of one dependent variable with the other. The variation of average depth (D) with water width (W) for both the cross-sections is shown in Figure-3 & 4. As to the situation at c/s G10 it can be seen that for widths upto about 2800m average depth is increased and for the widths larger than that it is decreased in 1999-00 compared to the 1998-99 situation. On the other hand at c/s G11 the average depth is reduced in 1999-00 for about all widths in comparison with that of 1998-99. The temporal relationships between area (a) and elevation (E) found for c/s G10 and c/s G11 are as follows;

C/S G10:	In 1998-99	$E = 0.1716\,A^{0.4256}$	$R^2 = 0.9992$
	In 1999-00	$E = 0.0914\,A^{0.4924}$	$R^2 = 0.9975$
C/S G11:	In 1998-99	$E = 0.0.0006\,A^{0.9986}$	$R^2 = 0.8924$
	In 1999-00	$E = 0.0138\,A^{0.6802}$	$R^2 = 0.9874$

It is seen that at c/s G10 in 1999-00 situation a relatively smaller elevation is required to have the same area of 1998-99 situation for cross-sectional areas upto about 13000 m^2. However, for higher cross-sectional areas a reverse trend is noticeable. As to the situation at c/s G11 it is to be seen that for cross-sectional areas upto about 19000 m^2 a higher elevation is required in 1999-00 in order to have the same cross-sectional areas of 1998-99 situation and a reverse trend is noticeable for cross-sectional areas higher than 19000 m^2.

The Gorai

The variation in the suspended sediment transport with discharge at Kamarkhali appears in Figure-5. It is noticeable from the figure that a decrease in the suspended sediment transport for flow discharges upto about 2500 m^3/s and an increase for discharges higher than that have been occurred in 2000 compared to the situation in 1998. However, in 2001 suspended sediment transport is increased for discharges upto 3000 m^3/s and decreased for higher discharges compared to the situation in 1998.The temporal variation of the water levels with discharges at Kamarkhali is shown in Figure-6. It is revealed from figure that a drop in the water levels occurred in 2000 and 2001 compared to the situation in 1998.

The variation of mean bed levels for different discharges at Kamarkhali is shown in Figure-7. A fall in the mean bed level observed in 2000 compared to that of 1998. However, in 2001 situation the mean bed levels are seen to have increased again and for discharges higher than about 700 m^3/s it even exceeds the 1998 levels. The relationships between discharge (Q) and average depth (D) for the considered years appear below;

In 1998: $D = 0.402\ Q^{0.3213}$ $R^2 = 0.9533$

In 2000: $D = 0.5364\ Q^{0.2793}$ $R^2 = 0.9206$

In 2001: $D = 0.768\ Q^{0.2269}$ $R^2 = 0.8492$

It is seen that at discharges less than 1000 m^3/s the average depth (D) is increased with time whereas a reverse trend is noticeable for discharges higher than 1000 m^3/s.

The plot of discharge (Q) vs. water width (W) at Kamarkhali appears in Figure-8. It can be seen from figure that at Kamarkhali water widths are increased in 2000 for discharges upto about 2100 m^3/s and decreased for higher discharges compared to the 1998 situation. However, in 2001 situation water widths are reduced from the 2000 situation for all discharges.The plot of discharge (Q) vs. average velocity (V) at Kamarkhali shows the following temporal relationships between the two parameters;

In 1998: $V = 0.046\ Q^{0.4037}$ $R^2 = 0.9685$

In 2000: $V = 0.0232\ Q^{0.4975}$ $R^2 = 0.9744$

In 2001: $V = 0.021\ Q^{0.5246}$ $R^2 = 0.9735$

It can be seen from the relationships that for discharges upto about 1500 m^3/s average velocity is decreased and it is increased for higher discharges in 2000 compared to 1998 situation. In 2001 the average velocity is increased for about all discharges compared to the 2000 situation.

The relationships between discharge (Q) and cross-sectional area (A) at Kamarkhali for different years are shown below.

In 1998: $A = 21.693\ Q^{0.5965}$

In 2000: $A = 43.05\ Q^{0.5025}$

In 2001: $A = 47.533\ Q^{0.4754}$

It is interesting to notice that for discharges upto about 1500 m^3/s cross-sectional area has been increased and it has been decreased for higher discharges in 2000 compared to 1998 situation. In 2001 the cross-sectional area is decreased for all discharges compared to the 2000 situation.

The Mathabhanga

The variation in the suspended sediment transport with discharge at Hatboalia is shown in Figure-9. It can be seen from the figure that suspended sediment transport is increased in 2000 compared to the situation in 1998 in 1999. The temporal variation of the water levels with discharges at Hatboalia is shown in Figure-10. It is revealed from figure that water levels have dropped in 2000 compared to the situation both in 1998 and 1999.

The variation of mean bed levels for different discharges at appears in Figure-11. The mean bed levels are seen to have dropped from 1998 to 2001.

The relationships between discharge (Q) and average depth (D) for different years appear below·

In 1998: $D = 0.6736\, Q^{0.3279}$ $R^2 = 0.9739$

In 2000: $D = 0.7152\, Q^{0.2952}$ $R^2 = 0.9308$

In 2001: $D = 0.6927\, Q^{0.2866}$ $R^2 = 0.9752$

It can be seen from the relationships that average depth is reduced in 2001 for all discharges compared to the situations in 1998 and 2000.

The plot of discharge (Q) vs. water width (W) for Hatboalia appears in Figure-12. It can be seen from the figure that the water widths have been decreased in 2000 in comparison with those in 1998 for all discharges. However, in 2001 situation water widths are seen to have increased for very smaller discharges (<30 m^3/s) and further reduced for higher discharges.

The plot of discharge (Q) vs. average velocity (V) for Hatboalia shows that the average velocities have been increased in 2001 in comparison with those both in 1998 and 2000 for almost all discharges.

The grain size analysis of the bed material of the rivers shows the bed of the river Ganges upstream and downstream of the Hardinge Bridge mainly consists of sand particles with small amount of silt. In case of suspended sediment load it is noticed that size distribution varies with time within a single year. During lean period the percentage of sand particle decreases and the percentage of silt particle increases. The reverse trend is noticeable during flood season. It perhaps happens due to the fact that at high discharges a part of the bed material discharge goes into suspension resulting in an increase in the percentage of coarser material.

In the study temporal variation in the suspended sediment transport with discharge is shown for the river Ganges, Gorai and Mathabhanga. The temporal variation of the hydro-morphological variables with discharge has also been shown. If other independent variables remain more or less constant during the time period considered the temporal variation in the hydro-morphological parameters might have caused due to the variation in the suspended sediment transport. As to the situation of the river Gorai the temporal variation of the variables as revealed from the study has much to do with the effects of dredging at the Gorai mouth. The boundary conditions at the offtake may also vary during the time period taken into account. Therefore, it is not fair to state the temporal variation in the hydro-morphological variables occurred only due to variation in the suspended sediment transport. However, the influence of variation in the suspended sediment transport can not be undervalued.

The results obtained for the river Ganges is indicative of the fact that a large variation in the suspended sediment transport can force the river channel to undergo substantial changes in its shape. A large increase in the suspended sediment load will cause large siltation in the river channel resulting in a decrease in the average depth and a rise in the mean bed level. It would cause widening of the river and consequent bank erosion.

The results obtained from the analysis of the data of the river Mathabhanga clearly indicate what may be the immediate response of the river to an increase in suspended sediment transport. It is seen from the analysis of the data that with the increase in the suspended sediment transport the river makes attempt to transport the increased load by increasing the flow velocity. An increase in the flow velocity means an increase in the water surface slope. Therefore, a drop in the water level is inevitable.

It is to be mentioned here that a river undergoes complicated morphological processes due to variation in its independent variables and also due to human interference and tries to reach a Quasi-equilibrium state.

Conclusions and recommendations

Based on the outcomes of the study the following conclusions have been drawn:

- Variation in the suspended sediment transport has much to do with the morphological changes in the rivers

- A sudden increase in the suspended sediment transport may cause major changes in the hydraulic geometry of the river downstream

- If the suspended sediment transport exceeds the transport capacity of the river the reduction in the water depth would be caused by deposition of sediment

- For the river Mathabhanga the immediate response of the river to an increase in the suspended sediment transport is recognized by a decrease in the width and depth, drop in the water level and an increase in the flow velocity

- The river undergoes complicated morphological changes due to variation in the discharge and sediment transport

The following recommendations are put forward based on the study:

- In order to develop the vast water resources of Bangladesh the morphological processes that the rivers are undergoing should be recognized regularly

- The present hydraulic and hydrological data collection should be improved and extended

- Detailed study of morphological processes of the important rivers with wide range of data series is recommended

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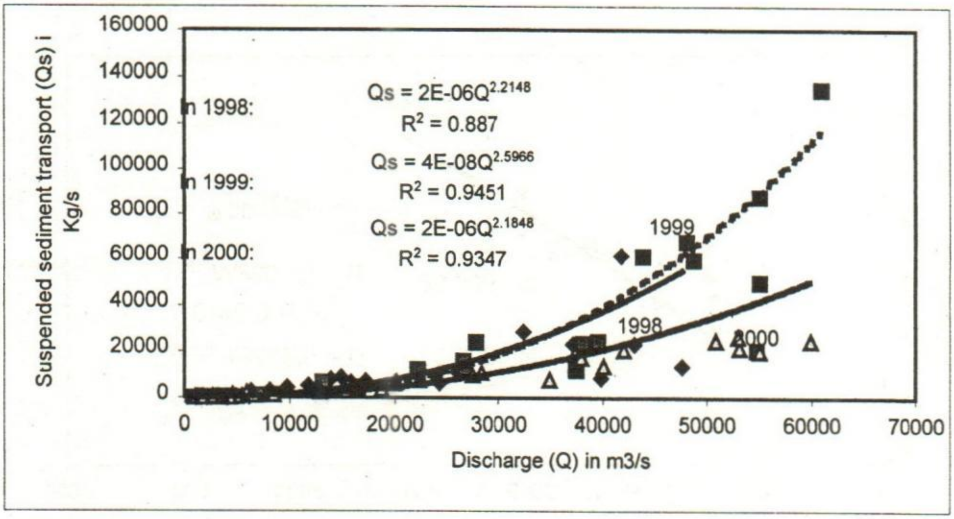

Figure 1: Temporal variation of suspended sediment load (Qs) with discharge (Q) of the river Ganges at Hardinge Bridge

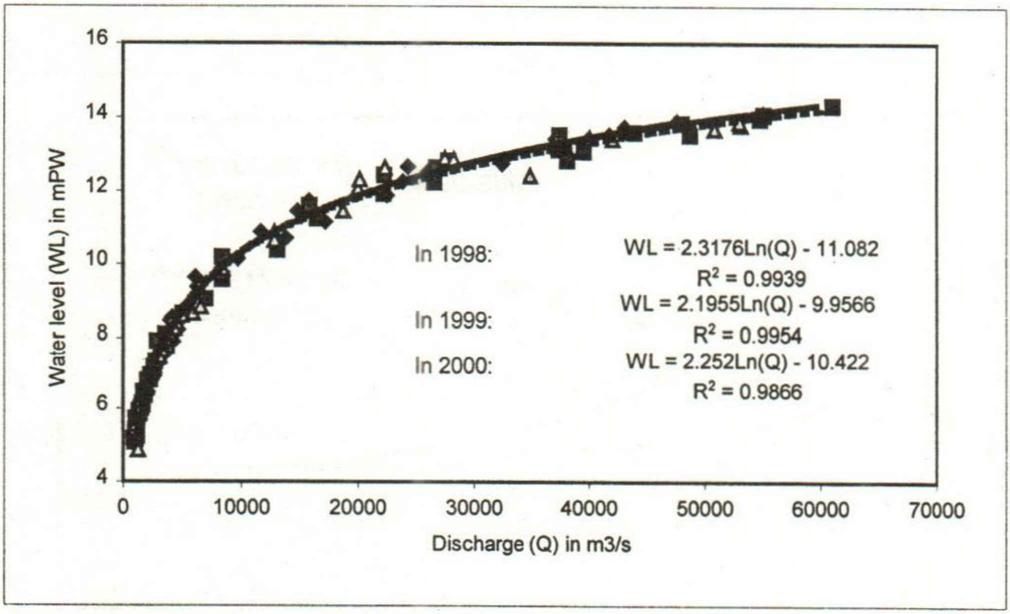

Figure 2 : Temporal variation of water level (WL) with discharge (Q) of the river Ganges at Hardinge Bridge

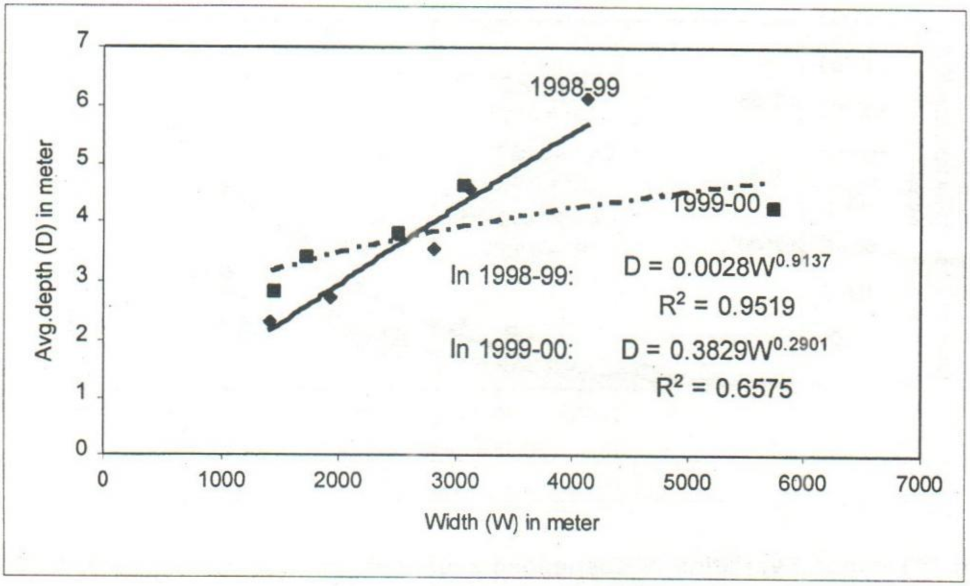

Figure 3: The variation of average depth (D) with water width (W) of the river Ganges at c/s G10

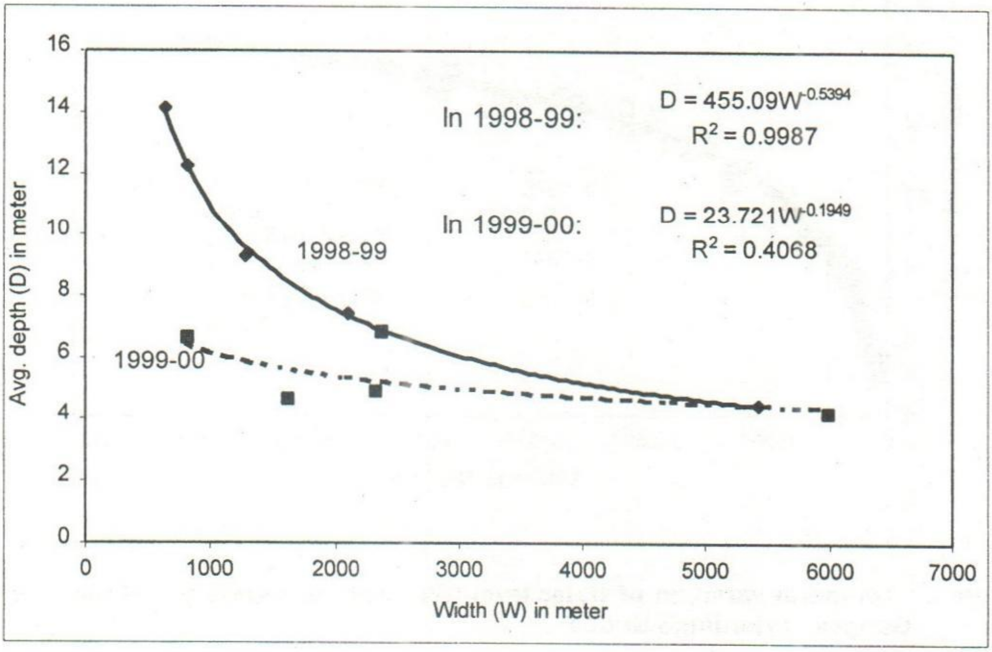

Figure 4: The variation of average depth (D) with water width (W) of the river Ganges at c/s G11

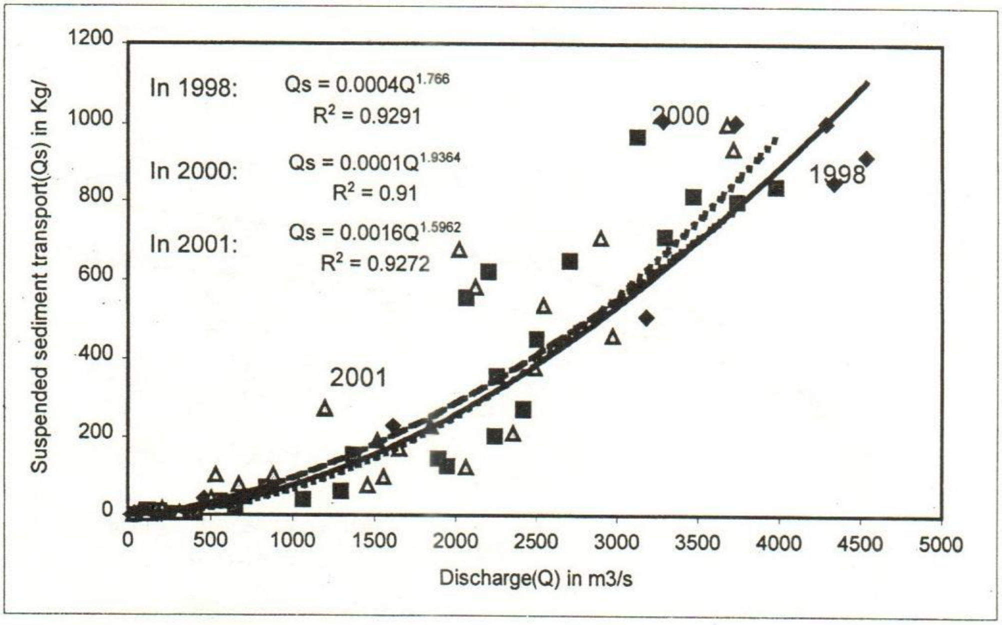

Figure 5: Temporal variation of suspended sediment transport (Qs) with discharge (Q) of the river Gorai at Kamarkhali

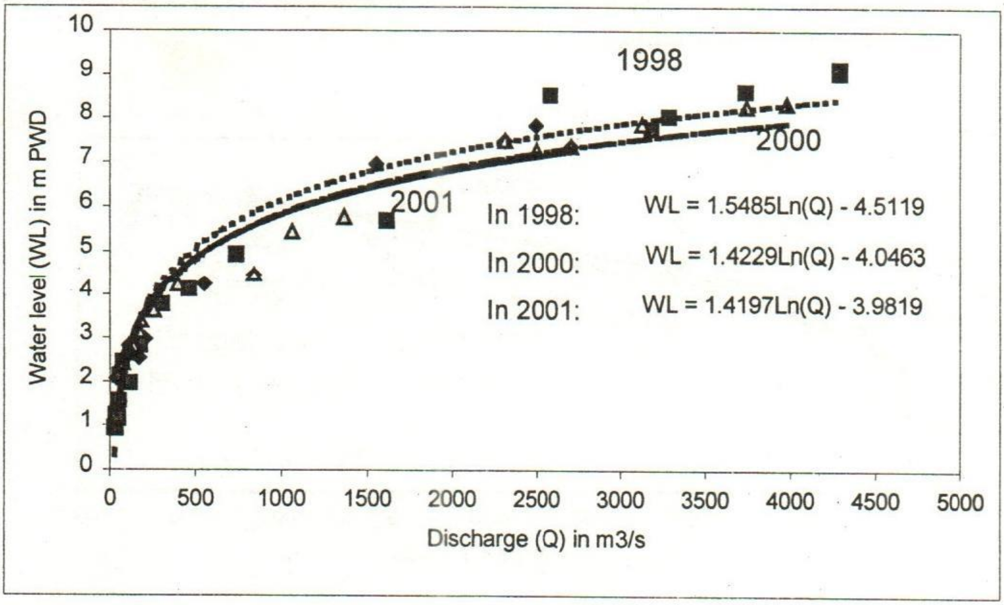

Figure 6: Temporal variation of water level (WL) with discharge (Q) of the river Gorai at Kamarkhali

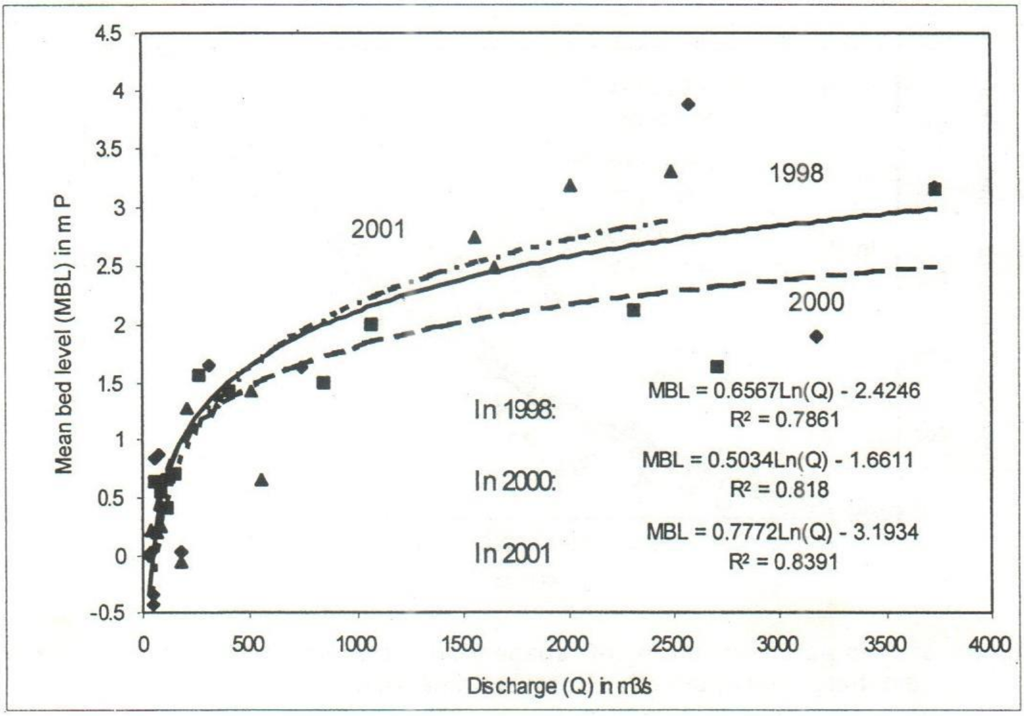

Figure 7: Temporal variation of mean bed level (MBL) with discharge (Q) of the river Gorai at Kamarkhali

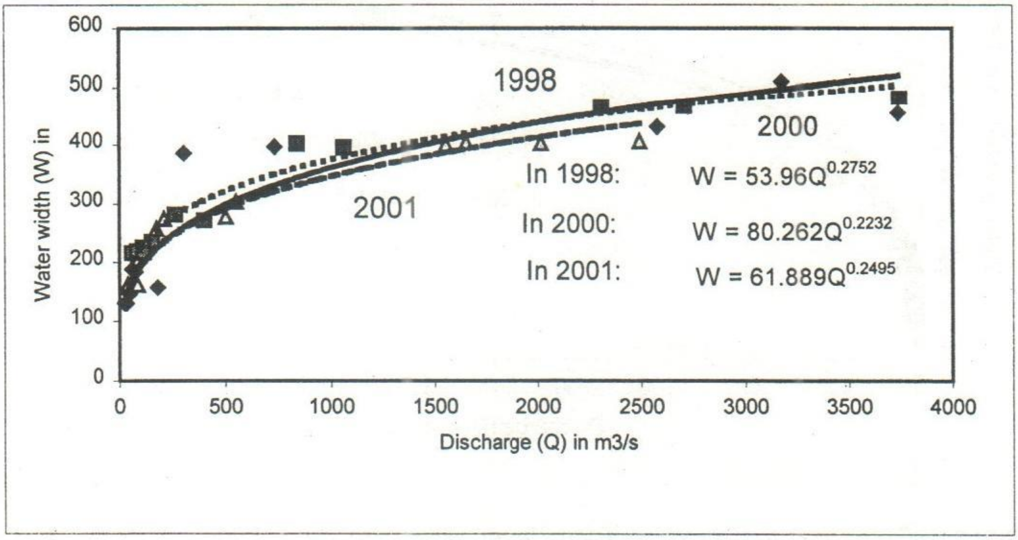

Figure 8: Temporal variation of mean bed level (MBL) with discharge (Q) of the river Gorai at Kamarkhali

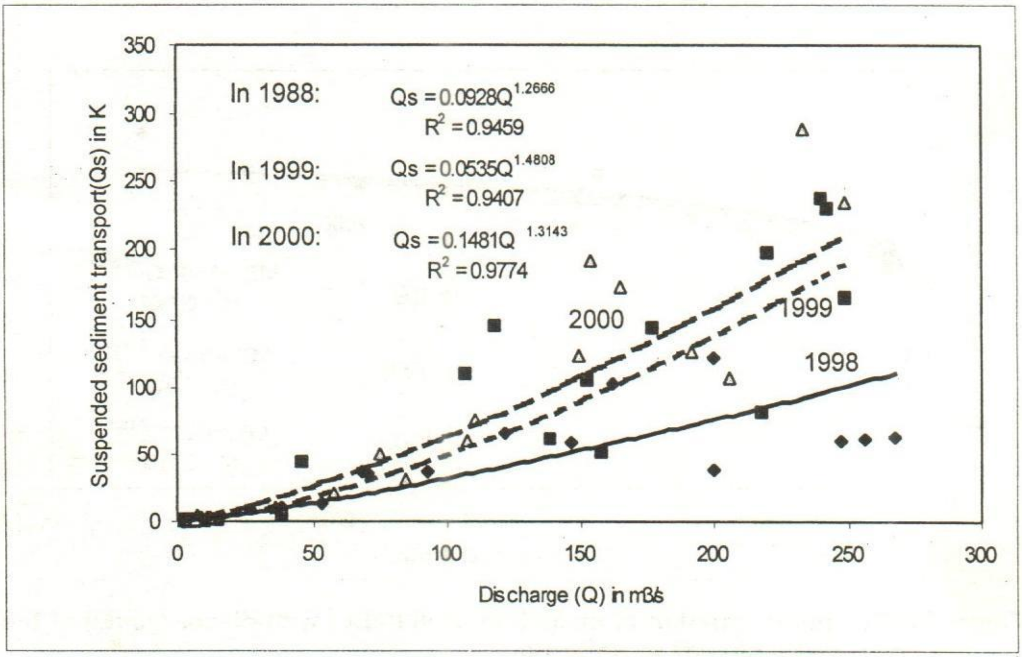

Figure 9: Temporal variation of suspended sediment load (Qs) with discharge of the river Mathabhanga at Hatboalia

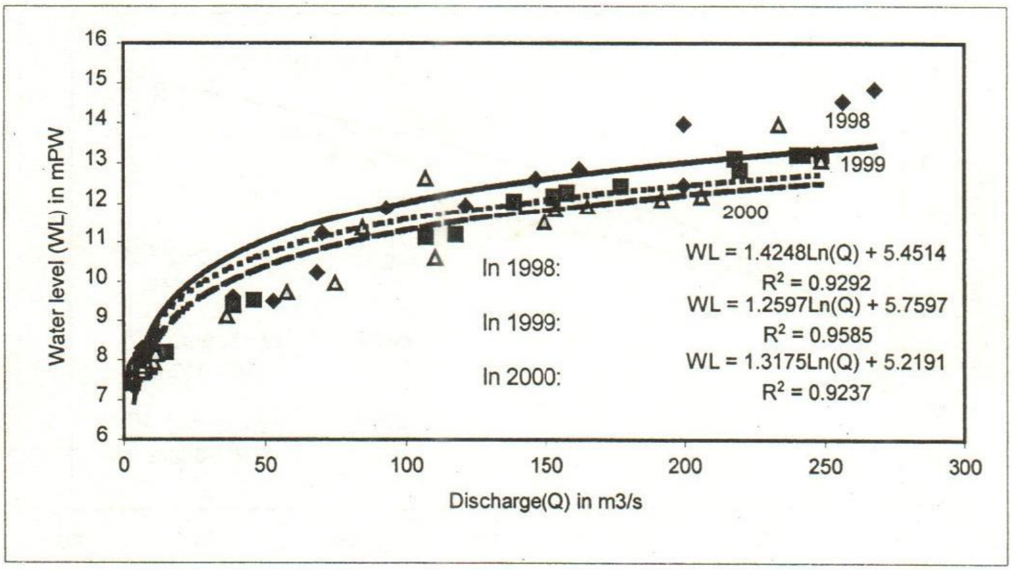

Figure 10: Temporal variation of water level (WL) with discharge (Q) of the river Mathabhanga at Hatboalia

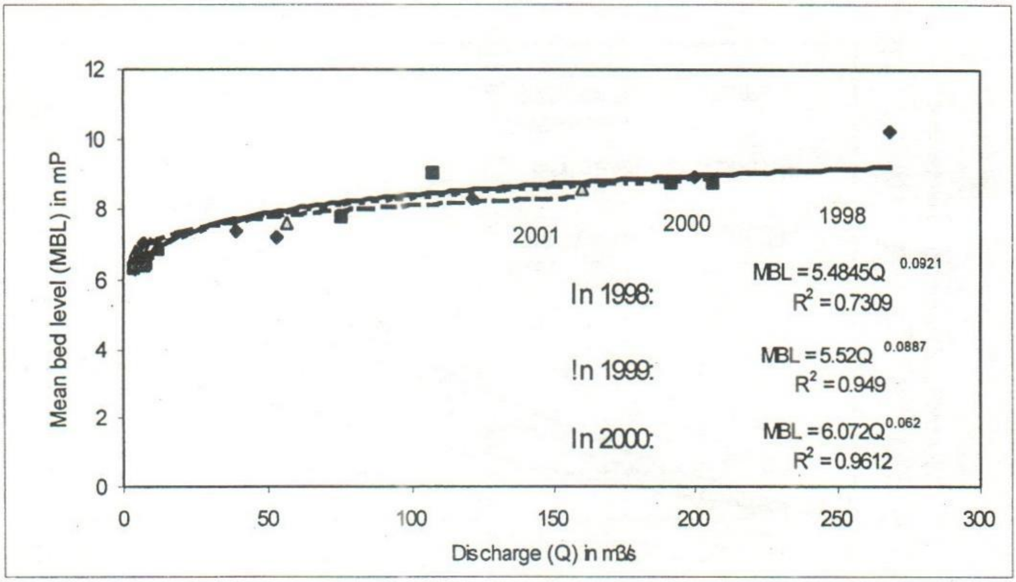

Figure 11: Temporal variation of mean bed level (MBL) with discharge (Q) of the river Mathabhanga at Hatboalia

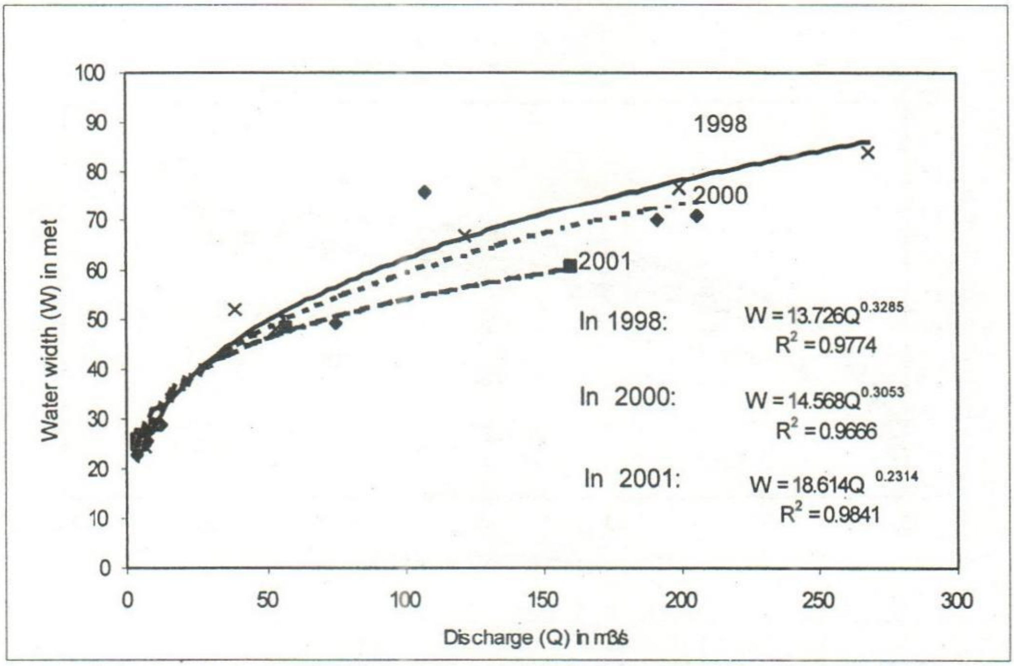

Figure 12: Temporal variation of water width (WL) with discharge (Q) of the river Mathabhanga at Hatboalia

Tech. j. River Res. Inst. 9(1): 57 - 64, 2002 (December)

STABILITY EVALUATION OF RAJSHAHI CITY PROTECTION EMBANKMENT AT KHOJAPUR AREA, RAJSHAHI, BANGLADESH

Younus Ahmed Khan[1], Takuo Yamagami[2] and Jing-Cai Jiang[3]

Abstract

Stability analyses and evaluations of a slope of Rajshahi City Protection Embankment (RCPE) at Khojapur area indicate some sort of potential failure possibilities due to critical horizontal earthquake accelerations (a_c) and softening effects of soil materials. Safety analyses were done in two different computer programs. First, the four different conventional slope stability analysis methods are used for factor of safety and then a progressive failure analysis program is applied to the specified critical failure surface. The RCPE has factor of safety of 1.421 to 1.408 at the high water level. Analyses relating to softening effects of the embankment show significant reduction of factor of safety. Considering the earthquake occurrence possibilities, the safety factors have been calculated with the different horizontal earthquake acceleration (a) at same water level. The calculated results (Factor of safety=1.003 to 1.012 with corresponding a_c=0.235g to 0.190g) show potential threats to the safety of the embankment at Khojapur area. The RCPE at Khojapur may fail at the horizontal earthquake acceleration of 0.19g.

This finding strongly recommends a detail investigation of seismic stability and softening phenomena of the RCPE

Introduction

The studied embankment is situated on the southern boundary of Rajshahi City in the Northern Bangladesh (Fig. 1). About 15 km long Rajshahi City Protection Embankment (RCPE) on the river Ganges is an important and costly engineering structure that provide an overall infra-structural protection to about 1 million city dwellers. The safety analysis for such type of embankments always needs careful and rigorous analytical techniques. This is an earthfill embankment consists fine-grained sands with silts and clays. Weathering and Soil erosion have been noticeably intense in recent years specially, after massive plantation on the embankment. The heavy monsoonal rainfall in the upstream sometimes increases the river water above the danger level. Such type of situation can threaten the embankment. In 1998 and 1988 the situation were worst where a number of water seepage had been developed along the city side slope of the embankment. Due to these weathering-erosion interaction and seepage action, there may develop strain softening of soil of the embankment. Strain softening drastically reduces the shear strength of soil materials from peak to residual value. Moreover, earthquake generates horizontal acceleration, which may reduce the shear strength considerably. The reduction of shear strength eventually induces failure along a critical failure surface of the slope. The Safety analyses of the river embankments have been

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pursued since the development of modern limit equilibrium technique (Fellinious, 1936; Bishop, 1955; Spencer, 1967; Junbu, 1954; Morgenstern and Price, 1965; Sarma, 1979).

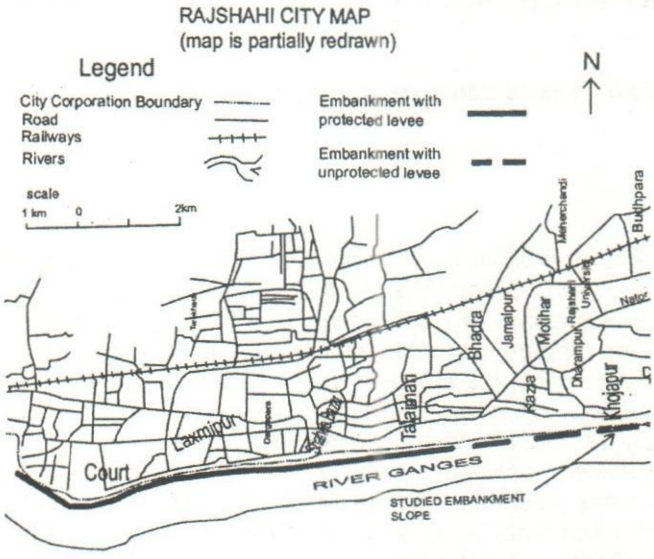

Figure 1. Rajshahi City map showing the studied slope location.

Any failed slope generally exhibits the behavior of progressive failure which states that local yielding or failure initiated at some points gradually develops and finally leads to overall failure of the slope along a slip surface. Such type of progressive failure is usually common in the river bank embankment and is rarely understandable with the above mentioned conventional methods of slope stability analysis. Recently, effective methods have been developed for the analyses of progressive failure (Yamagami and Taki, 1997; Yamagami et al. 1999, Khan et al. 2002). One of these methods of progressive failure analysis is used here along with other conventional methods.

There has been no significant published report on safety analysis of RCPE. Rahman (1988) performed some safety analysis on this embankment and predicted average values of safety factors for the embankment slope as a whole without mentioning any specific slip surface. According to Rahman (1988) the average safety factor of RCPE is 1.26 with the Fellinius method and 1.38 with the Bishop method. In his analysis no water level was considered which seems to be the most critical factor for any embankment analysis. Apparently the embankment is safe without any external forces, but it may fail under doftening effects and or possible earthquake's shaking conditions. The real situation of safety under different softening levels and possible horizontal earthquake acceleration is of prime importance of any stability analysis for such important embankment.
Therefore, this paper attempts the stability evaluation of RCPE at Khojapur area with the effects of softening of soil materials and horizontal earthquake acceleration.

Methods of analysis

Location and selected cross sections of RCPE

The Court-Talaimari part of the RCPE has recently been rejuvenated, but the Talaimari-Khojapur part is lying almost unprotected. The embankment part at Khoiapur area is seems to be most vulnerable. The field observations along the embankment from Court

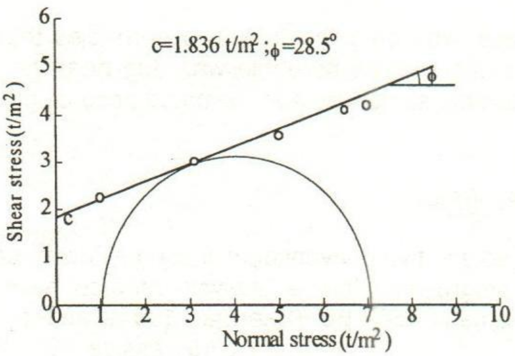

Figure 2. Failure envelope with Morh circle determined from shear box tests of the representative samples.

area to Khojapur area during the last two flood year indicate that Khojapur part of the embankment is more prone to soil erosion which deserves to be analyzed carefully. The geometry of the embankment slope at Khojapur area was traced out with clinometer and measuring tapes and then plotted on graph paper accordingly.

Determining the strength parameters and water level

In order to analyze the safety of the RCPE, several samples were collected from the specified cross sections. The samples were used in the laboratory to determine the strength properties. Undrained Direct shear test (using ASTM standard) was adopted for the shear strength parameters. The detail description of the shear box apparatus and the test procedures are found elsewhere (e.g. Punmia, 1985). The samples were subjected to a shear force and subsequent rupture by increasing the horizontal force until failure was induced. This was repeated for several values of normal force. The normal and shear stresses were found by dividing the normal forces and shear forces, respectively, by the cross section of the shear box, Then, a failure envelope (Fig.2) was obtained by plotting the normal stresses and the respective shear stresses. Mohr circles were constructed on the plot and the angle of shearing resistance ($\phi°$) and the cohesion, *c* were obtained correspondingly. The unit weight of the representative samples were obtained with the water replacement method described by Punmia, 1985. The average

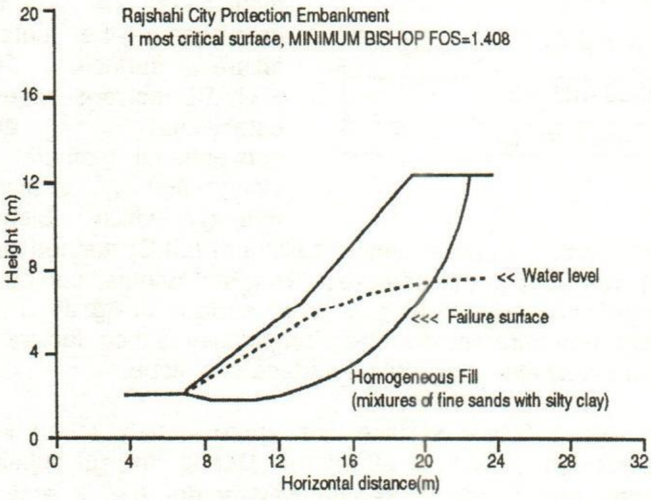

Figure 3. Slope geometry and XSTABL (Sarma, 1994) generated critical slip surface of RCPE at Khojapur.

unit weight of soil was 1.959 t/m^3. The average values of angle of shearing resistance and cohesion were determined as 28.5° and 1.836 t/m^2.

Within the studied slope, the water level was obtained from two boreholes that were used for installation of tube wells and one shallow borehole was dug near the toe of slope. With these data water level across the slope was approximated accordingly.

Factor of Safety calculations

Computer programs: XSTABL and ProgFan

In order to analyze the embankment's safety, five conventional limit equilibrium methods of slope stability analysis and one progressive failure analysis method have been chosen. Two computer programs namely XSTABL (Sharma, 1994) and ProgFan (Progressive Failure Analysis of slopes using Non-vertical slices) (Khan et al., 2002), were used for safety analysis. 'XSTABL', well-known computers program, consisting a group of slope stability analysis methods. XSTABL can handles searching techniques for determining the critical failure surface. The XSTABL includes several established and conventional methods of slope stability analysis among which Bishop method(1955), Spencer method (1967), General Limit Equilibrium (GLE) method and Junbu simplified method(1954) was used in the analyses. 'ProgFan' another computer program that can analyze progressive failure along a shear surface using the non-vertical slices within the limit equilibrium framework. ProgFan calculates local factors of safety as well as overall factor of safety along any failure surface of a slope.

Table 1. Calculated factor of safety with no horizontal earthquake acceleration a_c=0.0g)

	Methods	Factor of safety
XSTABL	Bishop method	1.408
	Spencer method	1.415
	GLE with *f(x)*	1.413
	GLE with half sine	1.412
	Janbu simplified method	1.421
ProgFan	Progressive failure analysis	1.419

In ProgFan, softening effects along failure surface are approximately taken into consideration in terms of peak and residual strengths. During the calculation procedures, ProgFan uses peak and residual strength parameters (i.e. ϕ and c) simultaneously. Later in the texts, the subscripts, p and r will be used for peak and residual strength parameters of soil respectively. Both the programs can take into consideration of a_c, but the softening effects with progressive failure only can be used in ProgFan.

Therefore, XSTABL was used here for finding constant overall factor of safety and on the other hand ProgFan was used for determining the variable local factor of safety as well as the overall factor of safety along the critical failure plane of the studied slope of RCPE.

Calculated results and discussions

The above mentioned two computer programs were used for the calculations of factors of safety of specified slope of RCPE. Using Bishop method in XSTABL program the critical failure surface was determined with the minimum factor of safety and is shown in figure 3.

The searched critical failure surface is in circular shape. Along this critical failure surface, other methods in XSTABL programs were used in order to find the factor of safety for individual method. The local factors of safety and the overall factor of safety for this same failure surface were determined with ProgFan. The calculated local factors of safety and the overall factor of safety are shown in the figure 4b. The factors of safety calculated from different slope stability methods using the XSTABL and ProgFan programs are shown in the Table 1.
It is noted here that the local factors of safety clearly indicate the possible failure zone along the shear surface.

Effects of softening on the safety of RCPE

Figure 5 shows the analytical results of several cases where the peak (ϕ_p & c_p) and residual (ϕ_r & c_r) strength parameters were treated differently. If we consider the shear strength of the soil materials of the embankment became reduced to a 80% of the peak value (case 2), the local failure zone will extend further and the overall factor of safety will reduce drastically ($F_{overall}$=1.33). Similarly for the case 3, where the residual strength became 50% of the peak strength, local failure zone dominates over the entire slip surface and the resulted overall factor of safety become close to 1.18 (Table 2).

Noticeably, such type of softening effects on the factor of safety is not possible to consider in the conventional slope stability methods. This exhibits a unique advantage of the method of progressive failure analysis.

Pseudo-static analysis of RCPE

The pseudo-static analysis for the embankment safety was done considering the horizontal earthquake acceleration (a) in terms of g (gravitational constant). First, the analysis was performed with XSTABL program and the results showed (Table 3) that the critical earthquake acceleration (a_c) for the mentioned section of the embankment ranges from 0.195g to 0.235g depending on different methods. It may be noted here that the horizontal earthquake acceleration which causes failure (i,e, F=1.0) in a slope is considered as critical horizontal earthquake acceleration

According to the results with ProgFan program, the slope may fail in response to a horizontal earthquake acceleration of 0.19g. At the critical value of acceleration (a_c) the local failure zone extends to almost entire shear surface and the failure is eminent.

In all the pseudo-static analysis the strength parameters was the same as shown in figure 6.

According to Newmark (1965) the critical acceleration of a potential failure mass is a simple function of the static factor of safety and the slope geometry and expressed as

$$a_c = (Fs - 1) g \sin\alpha \qquad \ldots \quad \ldots$$

where, Fs= static factor of safety; α=angle of slope.

Table 2. Values of overall factor of safety from progressive failure analysis (ProgFan) in three different cases of softening

Cases	Softening condition	Soil parameters	Foverall
Case 1	No softening $\phi_r = \phi_p$ $c_r = c_p$	ϕp=28.5°; cp=1.836 t/m2 ϕ_r=28.5°; c_r=1.836 t/m^2	1.419
Case 2	$\phi_r = 0.8\phi_p$ $c_r = 0.8c_p$	ϕp=28.5°; cp=1.836 t/m2 ϕ_r=22.8°; c_r=1.468 t/m^2	1.336
Case 3	$\phi_r = 0.5\phi_p$ $c_r = 0.5c_p$	ϕp=28.5°; cp=1.836 t/m2 ϕ_r=14.25°; c_r=0.918 t/m^2	1.185

Table 3. Values of ac for Khojapur section of RCPE from different methods of XSTABL and proposed method.

Methods	Acceleration, a_c	$F_{overall}$
Bishop method (1955)	0.22g	1.006
Simplified Janbu method (1954)	0.195g	1.003
Spencer method (1973)	0.235g	1.005
General Limit Equilibrium (GLE) method	0.235g	1.003
ProgFan	0.190g	1.012

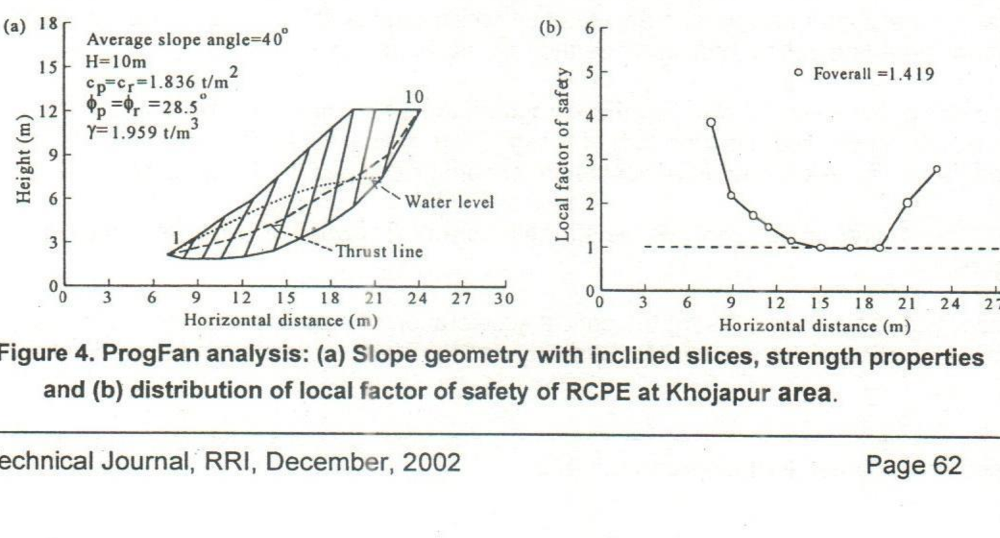

Figure 4. ProgFan analysis: (a) Slope geometry with inclined slices, strength properties and (b) distribution of local factor of safety of RCPE at Khojapur area.

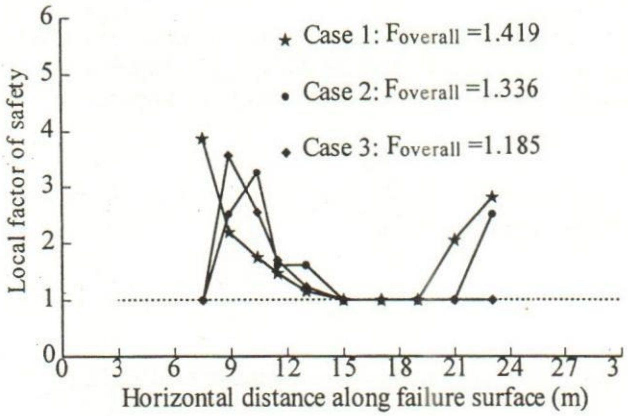

Figure 5. Influence of softening on local factor of safety

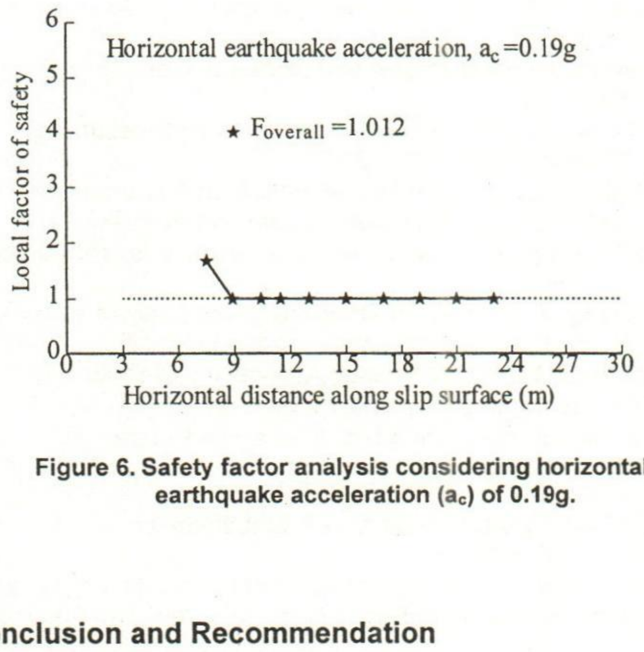

Figure 6. Safety factor analysis considering horizontal earthquake acceleration (a_c) of 0.19g.

Newmark (1965) pseudo-static above equation for critical acceleration was also considered in the present analysis. According to the equation 1, the critical horizontal earthquake acceleration determined for this case was 0.263g.

The value determined from the progressive failure analysis method (ProgFan) and those of the other methods differs a little. The value of a_c is less than the other methods, this means that the embankment slope of RCPE is supposed to fail at lower acceleration level and which is about 0.19g. Moreover local factors of safety are found so that local failure zone is evident in the ProgFan analysis. Again, the Newmark's equation shows higher acceleration than that of ProgFan. Therefore, the use of the progressive failure analysis with pseudo-static consideration is comparatively safer in the redesign works of the RCPE.

Conclusion and Recommendation

Stability evaluation of Rajshahi City Protection Embankment (RCPE) at Khojapur area using two programs of stability analyses (XSTABL and ProgFan) significantly indicate failure threats of the embankment slope due to softening effects and horizontal earthquake acceleration. Several conventional limit equilibrium methods and a progressive failure analysis method were used for the safety analyses. Constant overall factor of safety of the embankment slope was determined using the conventional methods and variable local factors of safety along critical failure surface were also calculated with a method of progressive failure analysis. Local failure zone along the slip surface was also evident at the studied slope. This embankment slope may fail due to strength reduction of soil with softening phenomena. Again, failure can occur at the earthquake's effects with critical horizontal acceleration ranges from 0.19g.to 0.235g.

This value of critical horizontal earthquake acceleration can be included in redesigning work of the embankment. However, water table information used in this study was not adequate for design purposes, it recommends detail water level investigation within the embankment slope.

The present paper strongly recommends detail investigation in redesigning the embankment slope of Talaimari-Khojapur part of RCPE.

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EFFECTS OF A SERIES OF SOLID SPURS AT A BEND ON FLOW FIELD: A CASE STUDY

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Mohammad Asadul Bari[3] and Md Kayser Habib[3]

Abstract

In the recent past solid spurs in a series have been constructed at many erosion prone areas of Bangladesh. The combined effects of such constructions are seen to be very well in stabilizing riverbank. Since solid spur is an active measure of bank protection it can modify the existing flow pattern. When constructed in a series at a bend to prevent the bend from migrating further they can have significant influence on the flow field at the bend. The thalweg position at a bend is very close to the bankline. Therefore, construction of such spurs extending into the river channel involves huge cost and also other difficulties. In mighty rivers like the Ganges and the Brahmaputra it is preferred to construct solid spurs in the floodplain. These spurs start functioning after a certain amount of bankline retreat between them. Scale model investigation can be an effective tool in determining the position, length, orientation and spacing of the spurs. This paper reports the effects of a series of proposed spurs to be constructed for the protection of Panka Narayanpur area of Nawabganj district on flow field. The spur locations have been determined from mathematical model investigation. The scale model study is employed to determine the likely extent of bankline retreat between the proposed spurs and consequent changes in the flow field. The outcomes from the study reveal the fact that substantial bankline retreat is likely to occur at pocket-2 (between spur 2 & 3) and pocket-3 (between spur 3 & 4) and the existence of the proposed flood control embankment might not be threatened unless any change occurs in the upstream boundary conditions.

Introduction

Panka Narayanpur area is situated on the left bank of the Ganges River in Chapi Nawabgang Sadar Upazila under Chapai Nawabgang district. The river is very erosion prone at this area. It has already eaten up vast cultivable land and homestead. A large number of government and public establishments are now under threat of erosion. So it is now an utmost need to protect the area from the severe erosion of the Ganges. Figure 1 shows the bank erosion situation at Panka Narayanpur. Under this circumstance BWDB has taken up a project to protect the area from bank erosion. RRI is given with the responsibility to conduct a physical model study for the project to investigate the efficacy of a series of proposed solid spurs in protecting the area.

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Accordingly a model is constructed based on the bathymetric data, bankline and embankment position supplied by SWMC (now IWM). On the other hand Design Circle-6, BWDB supplied the necessary design drawings of the proposed spurs. In addition to the base run a number of six application tests were conducted during the model study with maximum recorded flood discharge and corresponding bankfull discharge of 1988. It became possible from the model study to determine the amounts of retreat of bankline between different spurs and the maximum velocity at the head of the proposed spurs. All these information is expected to be useful to understand the consequence of the construction and to take necessary precautionary measures in design and construction of the spurs.

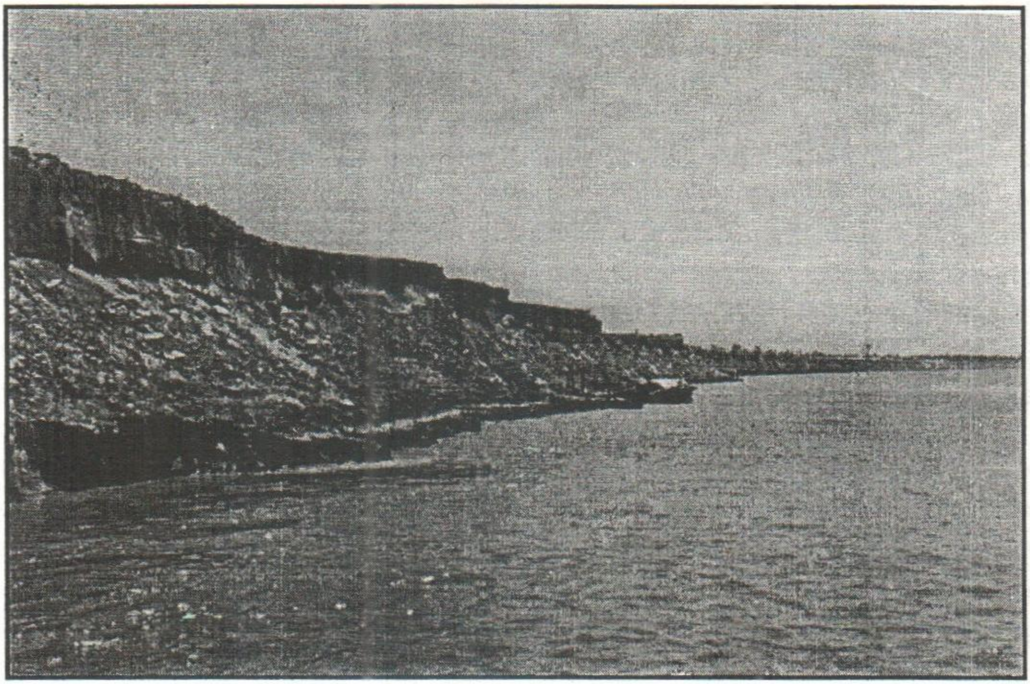

Figure 1: The bank erosion situation at Panka Narayanpur area in left bank of the Ganges.

Objectives of the study

The objectives of the study are:

- To investigate the present flow pattern without any training works.
- To investigate the effects of the proposed spurs on flow field.
- To investigate the possible bankline retreat between the spurs and consequent changes in the flow pattern.
- To determine the minimum distance between the retreated bankline and proposed embankment in different pockets.

METHODOLOGY

Study Approach

The scale model study is conducted keeping the following major ends in view

- Identification of the erosion prone areas at Panka Narayanpur.
- Determination of flow pattern with proposed spurs and primary identification of possible bankline retreat between the spurs.
- Schematization of the bankline retreat and investigation of flow pattern in the embayment.
- Investigation of changes in the flow pattern due to bankline retreat.

In order to meet the above requirements the study is planned to be conducted as a fixed bed flow model. In this regard geometric distortion of the model is needed not only to cover a length of about 20-km but also to fulfill the roughness condition of the model. It is to be mentioned here that in an undistorted sand-bed model it is almost impossible to fulfill the roughness condition.

Based on the bathymetric, hydrological and sediment data of October'2001 the horizontal and vertical scales of the model are selected as 1:280 and 1:100 respectively. The basic considerations behind the selection of these scales are:

- The available space for conducting model study at RRI.
- The available pumping capacity at RRI
- Fulfillment of roughness condition in the model.

Model design

The model is a distorted model with horizontal scale 1: 280 and vertical scale 1:100. The bed of the model is moulded with sand i.e. it is a sand bed model. From the selected geometric scales the scale for other parameters are determined and shown in the following table (Table1).

Table 1: Scale conditions for the different basic and derived parameters

Parameter	Unit	Scale
Velocity (V)	(m/s)	10
Time (T)	(s)	28
Slope (I)	(-)	0.357
Froude number (Fr)	(-)	1
Reynolds number (Re)	(-)	1000
Discharge (Q)	(m^3/s)	280000
Specific discharge (q)	(m^2/s)	1000
Chezy's coefficient (C)	($m^{0.5}$/s)	1.67
Manning's coefficient (n)	(s/$m^{0.33}$)	1.29

Non-fulfillment of roughness condition in the model may cause deviation in the above scales that should be noticed and taken into account during interpretation of test results.

Model set-up

The riverbed has been reproduced in the model according to the bathymetric survey data of October' 2001. An open-air model bed of 35m x 100m long has been used for setting up the model. The layout of the model appears in Figure 2. A standard sharp crested weir is used to measure the model discharge according to Rehbocks formula. A point gauge is installed upstream of the weir at a sufficient distance to avoid the effect of curvature during recording of the water level. Atmospheric air pressure is maintained below the nappe to reduce the drag effect with the help of a perforated pipe. A stilling pond is constructed to dissipate the energy of incoming flow and hollow bricks and bamboo screens have been used to adjust the distribution of flow at the inflow section. The water levels in the model at different locations are recorded with three point gauges. The water level in the model is controlled by operating tailgates constructed at the downstream end of the model. Provision is made to minimize erosion of the model bed during the filling operation.

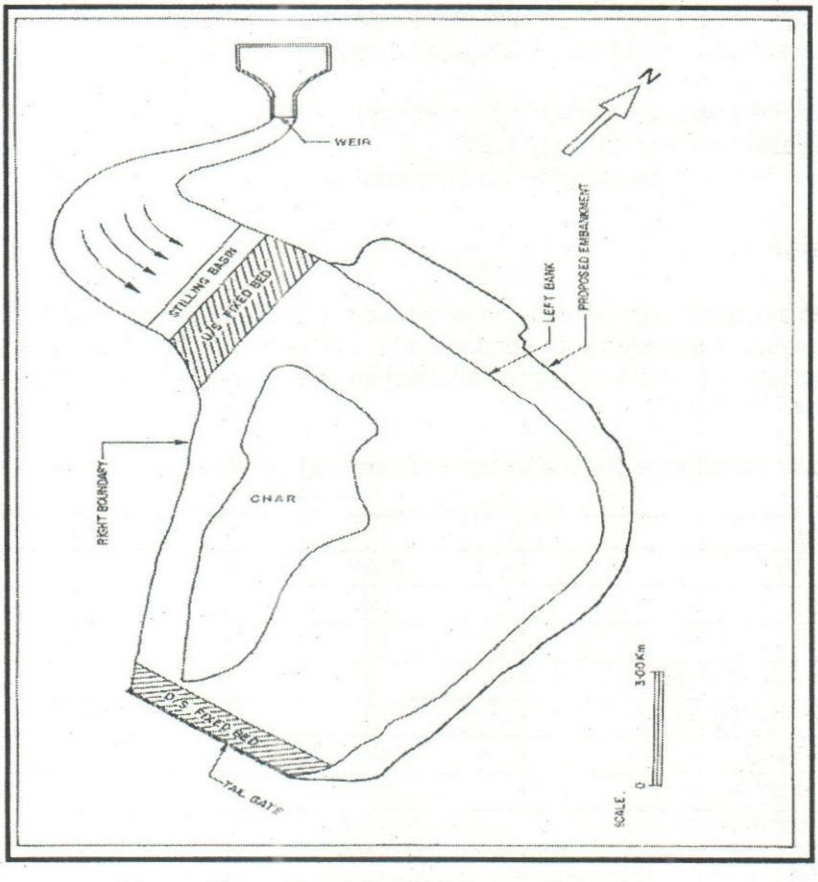

Figure 2: Layout of the model

Measurements

The necessary measurements that is made in the distorted fixed bed model are:

- Velocity distribution at c/s-121
- Discharge at an inflow section perpendicular to the flow
- Velocity distribution along some selected cross-sections
- Velocity around and at the head of the spurs
- Float tracks
- Water levels
- Near bank flow velocities
- Velocity in the embayment
- Amounts of bank retreat and distance between embayment edge and embankment
- Difference in water levels u/s and d/s of the spurs

Test scenarios

The test scenarios of the model study appear in the following table (**Table 2**).

Table 2: Test scenarios of the Panka Narayanpur model study

Test No.	Type of test	Test description	Discharge (m^3/s)
T01	Base run	Without intervention	71,656
T1	Application test	8(eight) BWDB proposed spurs	
T2	Application test	Retreat of bankline in pocket 2,3,4,5 and 6	
T3	Application test	Retreat of T2 bankline in pocket 2,3 & 5	
T4	Application test	Retreat of T3 bankline in pocket 2 &3	
T5	Application test	As in T4	32,207
T6	Application test	Retreat of T5 bankline in pocket 2	

Discussions on test results

During the study with the bathymetry and bank position of October'2001 a total of seven tests have been conducted including the base run. The results of each test and comparison of the results between different tests have been made. In the course of the study different measurements are taken systematically so that the effects of the proposed spurs can be investigated properly. From the analysis of the recorded data the following discussions are made as to the effects of the proposed spurs on flow field and the retreat of the bank line in different pockets in between the spurs. Discussions are also made regarding some other important aspects. The layout of the proposed spurs is shown in Figure 3.

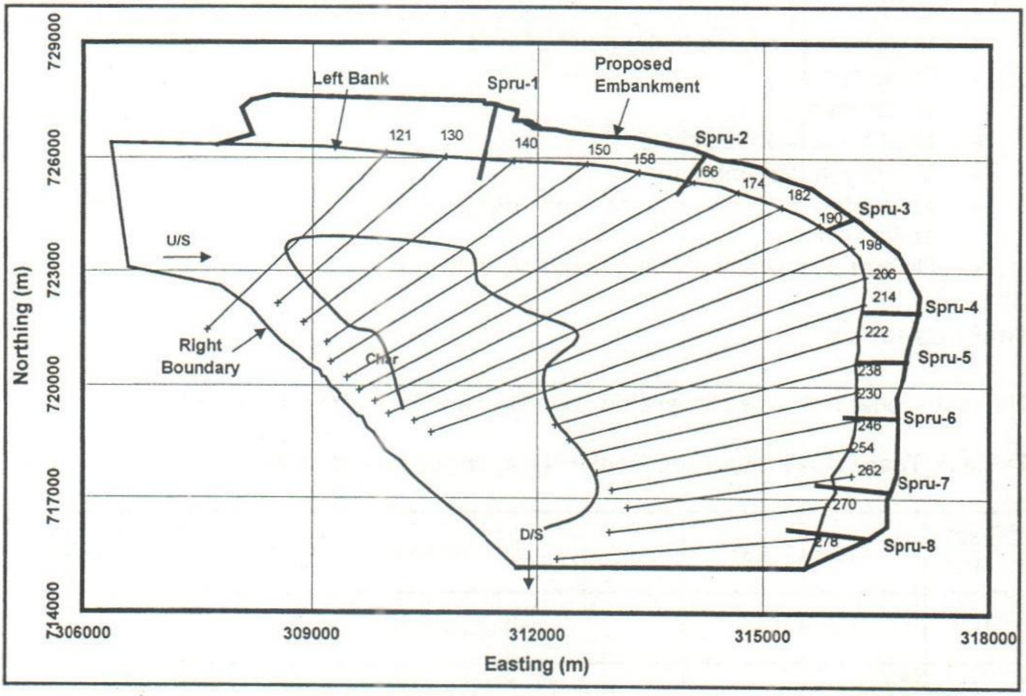

Figure 3: The position of spurs and cross-sections in model

Changes in the flow pattern

It is noticed from the test result that after construction of the spurs near bank flow concentration will be reduced from c/s-140 to c/s-218 and from downstream of c/s-234 to c/s-254. Elsewhere except at pocket-7 the near bank flow concentration will increase. It means the influence of the spur-1 and spur-2 extends upto the head of the spur-4. However, due to ineffectiveness of the spur-3 the reduction in the near bank flow concentration is not significant from c/s-178 to c/s-218. As a result the bank there is still vulnerable to erosion. The same holds for the river stretch from c/s-234 to c/s-254. It means that except pocket-1 and 7 entire bank could be subjected to erosion despite the presence of the spurs.

The third test (T2) is conducted with certain amounts of bank retreat in pocket-2, 3,4,5 and 6. The test results show that retreat of bank position in pocket-2 and 3 not only reduces the near bank flow concentration there but also improves the situation downstream. It mainly happens due to the fact that the spur-3, 4 and 5 that were almost inactive before now play some role in deflecting the oncoming flow away from the new bank. However, near bank velocity within a distance of 56 m from the bank still remains high enough to erode the bank.

The fourth test (T3) is carried out by further shifting the bank position leftward in pocket-2, 3 and 5. It is seen that it has caused more near bank flow concentration from downstream of c/s-190 to c/s-254. It points to the fact that a further shift in the bank position attracts more flow towards the bank near pocket-2 and 3 but the downstream spurs (spur-3, 4 and 5) are not effective enough to deflect all the oncoming flow well away from the bank. The flow lines for the test appear in Figure 4. It should be noted here that the above mentioned situation does not have any negative influence on the flow velocity very close to bank (within 56 m). On the other it is reduced noticeably compared to the T2 situation. However, its negative influence is observed on the magnitude of reverse flow velocity.

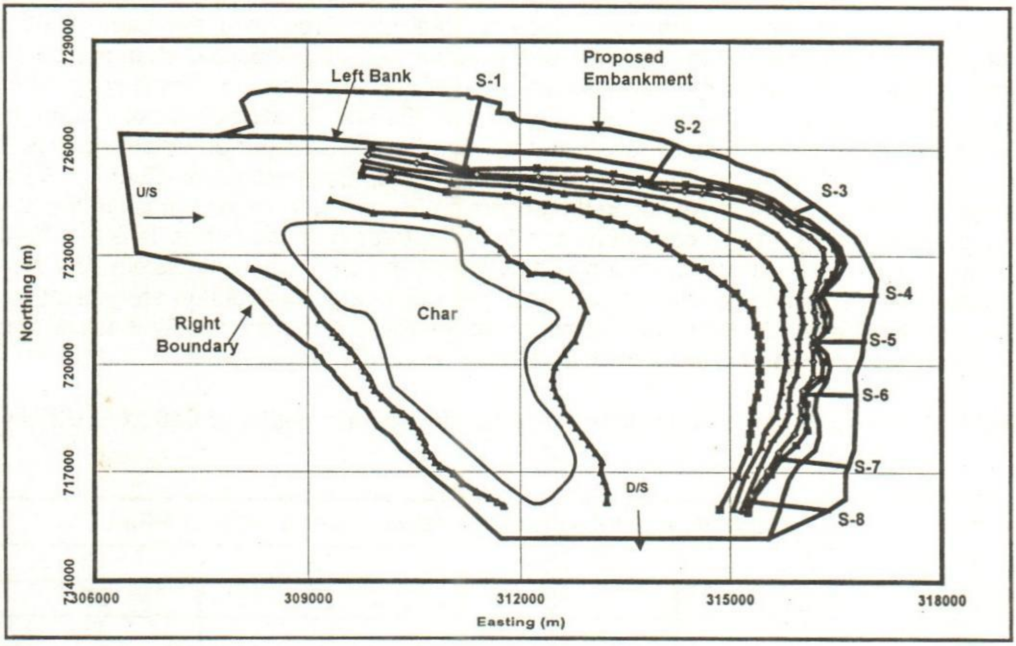

Figure 4: Flow lines in test T3

The fifth test (T4) is conducted by making more shift in the bank position in pocket-2 and 3. It is done because in the previous test near bank flow velocity there is found to be more than 1 m/s. The results from this test show that the magnitude of flow velocity very close to the new bank (within 56 m) is improved a bit. This velocity is now found to be less than 1 m/s.

The sixth test (T5) is conducted with a low discharge (bankfull discharge) to see the near bank velocity situation. During this test all other test conditions except discharge are kept the same as test T4. The test results show substantial increase in the magnitude of near bank flow velocity in pocket-2 whereas a bit increase is noticeable in pocket-3.

The seventh test (T6) aims to see the effects of a further retreat in the bank position in pocket-2 on near bank flow velocity. The test results show that such a retreat has resulted in a near bank velocity having magnitude less than 1 m/s in both the pocket-2 and 3.

It can be concluded from the outcomes of the above mentioned tests that the total amounts of bank retreat that has made during the study will result in a situation where near bank velocity will be insignificant or not so high to erode the bank.

Discharge distribution pattern

From the measured point velocities along different cross-sections the percentage of total discharge flowing over different widths extending from the left bank has been calculated. In case of bankline retreat the widths have always been measured from the new bank position. The variation in the percentage of total discharge over the same width in different tests indicates the influence of the spurs and bank position changes on flow field. Table 3 represents the percentages of total discharge flowing over a width of 840 m in different tests. The values that represent the T01 and T1 conditions point to the fact that beyond the limit of influence of Spur 1& 2 more flow has got attracted towards the left because of virtually no influence of Spur 3, 4 & 5 on flow. However, Spur 6,7 &8 are seen to have influence in deflecting the oncoming flow away from the bank. On the other hand values for T2 and T3 conditions signify that retreat in the bank positions in different pockets would only attract the nearby flow towards the bank whereas distant flow would remain almost unaffected. Erratic changes in the discharge distribution are indicative of the fact that development of embayments would activate the previously inactive spurs with consequent changes in the flow field.

Table 3: Percentages of total discharge flowing over a width of 840 m in different tests

C/S No.	Percentage of total discharge flowing over a width of 840m			
	T01	T1	T2	T3
178	26	23	29	27
186	31	27	24	19
190	30	22	17	23
202	31	27	23	27
210	37	29	24	27
214	36	33	27	24
218	35	32	32	37
222	35	38	19	33
226	33	35	29	33
230	32	33	31	37
234	34	35	25	30
238	30	28	25	28
254	31	27	18	24
262	26	27	20	17
270	33	31	31	29
274	36	35	21	27

Changes in the cross-sectional mean velocity

The mean cross-sectional velocities over different widths in different tests have been calculated. Table 4 shows the mean cross-sectional velocities over a width of 840 m from the left bank in different tests. It can be concluded from the values for T01 and T1 that introduction of spurs would cause a decrease in the mean cross-sectional flow velocity except from c/s-214 to some extent downstream of c/s-230. It confirms the findings as described in the previous section.

Table 4: Cross-sectional mean velocity over a near bank width of 840 m along some cross-sections in different tests

C/S No.	Cross-sectional mean velocity over a width of 840 m			
	T01	T1	T2	T3
178	2.08	1.79	2.26	2.04
186	2.30	2.16	1.71	1.33
190	2.21	1.81	1.37	1.53
202	2.46	2.02	1.91	1.84
210	2.81	2.18	1.83	1.86
214	2.40	2.58	2.13	1.87
226	2.32	2.71	2.15	2.43
230	2.23	2.52	2.45	2.73
238	2.37	2.28	2.14	1.87
254	2.45	2.20	1.58	1.98
262	2.54	2.38	1.82	1.54
270	2.58	2.39	2.40	2.55
274	2.60	2.52	1.82	2.47

Velocity at the head of the spurs

The magnitude of flow velocity at the head of the spurs measured during different tests is shown in the following table **(Table 5)**.

Table 5: Velocity at the head of the spurs measured during different tests

Spur No.	Q = 71,656 m^3/s				Q = 35,207 m^3/s	
	Test T1	Test T2	Test T3	Test T4	Test T5	Test T6
	Velocity (m/s)	Velocity (m/s)	Velocity (m/s)	Velocity (m/s)	Velocity (m/s)	Velocity (m/s)
1	3.44	3.35	3.24	3.20	2.90	2.89
2	2.02	1.98	2.08	2.12	2.56	2.51
3	1.30	1.91	2.36	2.08	3.27	3.31
4	2.36	2.93	3.34	3.34	3.07	3.16
5	2.02	2.08	2.22	1.88	1.58	1.67
6	3.71	3.81	3.85	4.19	3.10	3.15
7	4.25	3.41	3.54	3.10	2.49	2.48
8	3.78	4.20	4.30	3.98	4.20	4.21

The information presented in the table shows that initially spur-3,4 and 5 have at all no influence on flow during a high discharge. The influence of spur-2 is also not of so much significance. However, at a low discharge more flow concentrates on the head of the spur-2. It can be seen that in a high flow situation the flow concentration at the head of the spur-3, 4 and 5 increases with the retreat of bank position in pocket-2 and 3. When this retreat of bank position exceeds a certain extent the flow concentration at the head of the spur-3 and 5 again reduces whereas it remains unaffected at the head of spur-4. As to the situation at the head of the spur-6 it can be seen that the magnitude of velocity at the head of this spur keeps on increasing with the retreat of bank position in pocket-2 and 3. However, at the head of the spur-7 and 8 this trend can be described more or less as decreasing and increasing respectively. At a low discharge the flow velocity at the head of the spur-2 and 3 is seen to be high compared to the high discharge situation. It means during low discharge flow tends to direct towards the bank there. It is important to notice from the velocity magnitudes given in the table that at a low discharge the velocity at the head of the spur-5, 6 and 7 is less than that measured during the high discharge situation. However, at the head of the spur-8 it is comparable in magnitude with that measured at high discharge.

Retreat of bank line

During different tests different amount of bank retreat made at vulnerable bank position. The total shift in the bank positions along different cross-sections appears in (Table 6). It is important to note here that prior to a particular test bank positions have been retreated based on the flow lines observed in the previous test. The flow lines are seen to have assumed an elliptical shape. So the bank retreat is done accordingly. It can be seen from the table that maximum bank line retreat has to make along c/s-206 and c/s-210. The river bank is seen to be most vulnerable at pocket-2 and 3. Therefore, major bank line shift is required there.

Table 6: Total shift in the bank positions along different cross-sections

C/S No.	Total bank position retreat (m)
178	98
182	294
186	275
190	205
198	168
202	280
206	420
210	448
214	350
222	98
226	188
234	154
238	207
242	140
254	118
258	322
262	42

Minimum distance

The minimum distance is defined here as the minimum distance between the bank and the embankment at different pockets in different tests. It is to be mentioned here that due to retreat of the bank position the amounts of minimum distance is likely to vary at the same pocket in different tests. However, this minimum distance may not occur at the same location in all tests. The minimum distances measured at different pockets in different tests appear in the following table (Table 7). It is important to notice from the above information that minimum distance after the retreat of the bank line at pocket-2 and 3 is 184.8 m and 221.2 m respectively. It means the proposed embankment there is not quite safe from being eroded in case of occurrence of an unusually high flood event and also due to sudden changes in the upstream conditions of the river. However, the minimum distances found in the other pockets can be considered as safe if outflanking does not occur at pocket-2 and 3.

Table 7: Minimum distances between the bank and embankment at different pockets in different tests

Pocket No.	Initial minimum distance between embankment and bank line (m)	Minimum distance (m)				
		Q = 71.656 m3/s			Q = 35,207 m3/s	
		Test T2	Test T3	Test T4	Test T5	Test T6
1 (between spur 1 & 2)	728	No cut	No cut	No cut	No cut	No cut
2 (between spur 2 & 3)	468	333.2	291.2	240.8	240.8	184.8
3 (between spur 3 & 4)	426	322	310.8	221.2	221.2	221.2
4 (between spur 4 & 5)	661	456.4	456.4	456.4	456.4	456.4
5 (between spur 5 & 6)	546	421	361.2	361.2	361.2	361.2
6 (between spur 6 & 7)	566	515.2	515.2	515.2	515.2	515.2
7 (between spur 7 & 8)	661	No cut	No cut	No cut	No cut	No cut

Difference in upstream and downstream water level at different spurs

During the test upstream and downstream water levels are measured at the junction of the earthen shank and RCC part of each spur. From the measurements of water levels the difference between upstream and downstream water levels has been calculated for every spur. The results are shown in Table 8.

Table 8: Difference in upstream and downstream water levels at different spurs

Spur No.	Upstream and downstream water level difference (m)		
	Test T4 (Q=71,656 m^3/s)	Test T5 (Q=35,207 m^3/s)	Test T6 (Q=35,207 m^3/s)
1	0.5	0.4	0.4
2	0.4	0.3	0.3
3	0.2	No water	No water
4	0.3	0.3	0.3
5	0.2	0.3	0.3
6	1.1	0.7	0.8
7	0.6	0.2	0.2
8	1.2	1.2	1.2

From the information presented in the table it can be seen that the water level difference is greater than 1 m at spur-8 for the two different discharges whereas at spur-6 the difference is greater than 1 m only for a very high discharge.

Velocity around the spurs

During the tests the magnitude of flow velocity around the spurs has been measured. The measured velocities indicate that at the very beginning i.e. when no bank retreat occurs at any pocket the RCC part of spur-6, 7 and 8 will be under tremendous thrust of oncoming flow. On the other hand the same will experience moderately high thrust at spur-1. After a certain amount of bank retreat at different pockets (test T4 situation) heavy thrust of oncoming flow on RCC part will occur at spur-6 and 8 whereas the same of the spur-1, 3,4, 5 and 7 will face moderately high thrust.

Near bank flow velocity

From the measurements of velocity during different tests it is seen that the entire river reach is under threat of erosion without any protective works. After the introduction of the proposed spurs erodible near bank velocity occurs at all pockets except pocket-1 and 7. However, certain amount of bank retreat will improve the situation at pocket-4, 5 and 6 and no more erodible velocity will occur there within a distance of 56 m from the left bank. On the other hand at pocket-2 and 3 a substantial amount of bank line retreat will only ensure occurrence of a low magnitude (< 1 m/s) near bank velocity.

Conclusions

The study is carried out to investigate the performance of eight BWDB proposed spurs to protect the Panka Narayanpur area from the erosion of the river Ganges. The position, length and orientation of all the spurs are prefixed and there is no scope to change any of these in the model study. The model investigation is intended to see the effects of the proposed construction on flow field and also on the likely bank erosion in between the spurs. Therefore, the outcomes of the study provide comprehensive information regarding near bank flow concentration, near bank velocity, velocity around and at the head of the spurs etc. on the one hand and regarding amounts of likely bank line retreat at different pockets on the other. The following conclusions are drawn from the outcomes of the study hand:

- The entire river reach is under threat of erosion without any protective measure
- The introduction of the proposed spurs will not prevent the bank at pocket-2, 3,4,5 and 6 from being eroded if no additional protective measure is taken to this end.
- A certain amount of bank erosion will improve the situation at pocket-4, 5 and 6 in terms of magnitudes of near bank (new bank) velocity.
- At pocket-2 and 3 a substantial amounts of bank retreat (maximum 468 m and 586 m respectively) from the original bank position is necessary to have a near bank velocity less than 1 m/s.
- In order to have a near bank velocity on the order of less than 1 m/s the minimum distances left between the bank and the embankment at pocket-2 and 3 are 184.8 m and 221.2 m respectively.
- Without any retreat in the bank line at pocket-2,3 and 4 the effect of the spur-3,4 and 5 on flow is almost negligible.
- With the progress of bank erosion in pocket-2 and 3 the magnitude of reverse flow velocity in the downstream pockets will increase.
- At a very high discharge the RCC part of spur-1, 4,6,7 and 8 will be subjected to heavy thrust of oncoming flow. Initially this thrust will be moderately high at spur-4 and very high at spur-7 but with the retreat of bank line at pocket-3 the thrust will increase at spur-4 and decrease at spur-7.
- At a low discharge (bankfull discharge) the magnitude of velocity at the head of the spur-2 and 3 is higher than that at a high discharge.
- For both a high and a low discharge maximum difference between upstream and downstream water levels occurs at spur-8 and it is 1.2 m.
- Retreat of bank position at pocket-2 and 3 attracts the nearby flow towards the new bank but it has little impact on the distant flow.
- The magnitudes of flow velocity at the head of the spur-1, 6,7 and 8 indicate that large scour hole will form near the head of those spurs.
- High magnitude of flow velocity at the head of the spur-6, 7 and 8 occurs due to poor performance of spur-3, 4 and 5 in deflecting the oncoming flow.

As mentioned earlier the above conclusions have been drawn investigating the performance of a series of proposed spurs with predetermined specifications. One or several changes in those specifications would result in different findings and therefore different conclusions. Scale model investigation can be employed as an effective tool to determine the most suitable specifications that meet all the requirements. It is apparent from the present study that series of spurs can be fairly considered to prevent a river bend from being eroded.

Recommendations

Based on the findings from the study the following recommendations are made:

- The minimum distances between the bank and the embankment at pocket-2 and 3 are very less. Therefore, special precautionary measure should be taken there to protect the embankment and to prevent likely outflanking.
- The basic design and construction requirements should be fulfilled properly so that the RCC part of the spur-1, 6,7 and 8 can withstand the tremendous thrust of the oncoming flow.
- After completion of the construction of the spurs the future developments should be monitored for several years and quick measure should be taken to prevent any negative development.
- In case of taking any additional protective measure scale model investigation is recommended to determine the most suitable option to this end.

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REGIONALIZATION USING GENERALIZED EXTREME VALUE-PROBABILITY WEIGHTED MOMENTS METHOD: A CASE STUDY

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Abstract

Regionalization or regional flood frequency analysis is a technique whereby the information contained in the records from nearby gauging stations is Generalized and transferred to the sites with no records available. In this study, regional flood frequency analysis was done using Generalized Extreme Value-Probability Weighted Moments method. This method has been applied to the 92 catchments (stations) in Indonesia (48 catchments in Java and 44 catchments in Sumatra). In this method it is remarked that the scaled regional Extreme Value Type I distribution encapsulates the data of most of the 92 stations. There are 14 stations out of 92 stations where clear departure from the data points can be observed. The outcomes of this method are not astonishing because of 5 stations mainly follow Extreme Value Type III distribution and two stations mainly follow Extreme Value Type II distribution. For the other 7 stations, the likely reason may be they possess separate flood region, which has not been determined by the current methodology.

Literature Review

There are several methods for regionalization. Some of the regionalization methods are describe here.

Multiple regression analysis

Regionalization can be done by the application of multiple linear regression analysis (MLRA). A regression equation is derived using flow quantiles from the gauged sites as the dependent variables, and the rainfall & catchment characteristics (area, main stream length, slope, drainage density etc) as independent variables. From the regression equation, flow estimates are then computed and subtracted from the corresponding quantile (derived from the observed data) to obtain the residuals. The residuals are plotted on a map and regional boundaries are then drawn to group together catchments with residual values that have similar magnitude and sign. The regression analysis is then repeated for each new group of stations or sub-region and finally the results are generalized over all sub-regions.

The flow characteristics obtained from a gauging station are subject to two sources of variations: variation among sites due to differences in catchment characteristics and variation due to sampling time at the gauged sites. In the multiple linear regression method, the flow for a given frequency level can be related to catchment characteristics by the following equation (Greis & Wood, 1981):

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$$log\ Q_i = klog10 + alogA + blogB + \ldots \quad (1)$$
$$i.e.\ Q_i = 10^k A^a B^b \ldots \quad (2)$$

Where, Q_i is the index flow or quantile, *A* & *B* are the catchment characteristics and *k, a* & *b* are the coefficients in the regression model.

Riggs (1973) applied the regression method to three flood peaks (Q_2, Q_{25} and Q_{50}) to regionalize the Snohomish river catchment in Washington using two catchment characteristics (drainage area *A* in mile^2, and mean annual precipitation *P* in inches) and obtained the following models:

$$log\ Q_2 = -2.07 + 0.954\ logA + 1.96\ logP \quad (3)$$
$$log\ Q_{25} = -2.07 + 0.970\ logA + 2.11\ logP \quad (4)$$
$$log\ Q_{50} = -2.07 + 0.955\ logA + 2.16\ logP \quad (5)$$

Mimikou (1990) also applied regression analysis to regionalize catchments in Greece. She used the variables listed in **Table 1.**

Table 1. The variables used by Mimikou (1990) in Greece

Variable	Description	Unit
A	Drainage area	km^2
P	Mean annual rainfall	Mm
L	Length of channel	Km
H	Hypsometric fall	M
S	Average bed slope	m/km
SF	Stream frequency	km^{-2}
M51D	Intensity of 1-day rainfall of 5-yr return period	mm hr^{-1}
SI	Soil index	

Index flood method

Dalrymple (1960) first described dimensionless flood frequency distributions applicable to all drainage catchments within a homogeneous region, which were applied for most of the flood-frequency analyses made by the United States Geological survey (USGS) before 1965. This method consists of the following steps:

i An at-site flood frequency curve relating magnitude of flood to return period *T* is developed for each site within the homogeneous region.

ii. The ratio of the Q_T corresponding to return period *T* to the mean annual flood Q_{mean} for various return periods is computed for the same site.

iii. Compiling ratios for all stations and finding the median ratio for each return period.

iv. Plotting the median ratios against the return periods to produce a regional frequency curve.

FSR-NERC method

This method was adopted in the United Kingdom Flood Studies (NERC, 1975), which is a modification of the Dalrymple method. The annual maximum flows for each station in the homogeneous region were assembled in the form of Q_T/Q_{mean} together with the plotting positions appropriate to the Gumbel distribution expressed as reduced variates. The *i*-th smallest value in the record of *N* years is associated with $y=E\ (y_i,\ N)$, the expected value of the *i*-th smallest order statistic in a sample size *N* from the reduced Gumbel variate. If the records were of the same length, the *i*-th smallest values would have the same plotting position. The length of records used in the Flood Studies Report (FSR) was not the same. So a simplification was made by dividing the reduced variate axis, *y* into categories of width *0.5* units (*e.g. –2.0 to –1.5, -1.5 to –1.0, -1.0 to –0.5 etc.*). All points that would be plotted in a given category were replaced by the average value of Q_T/Q_{mean} plotted at the average value of *y*. Since the resulting plot only covered the range from $y=-1.5$ to $y=3.5$ or *4.0*, extension to $y=5.5$ to *6.0* (*T=200 to 400* years) was made in the following manner. The stations in the region were divided into four or five groups with the requirement that neighboring stations do not appear in the same group in order to have statistically independent samples. The four highest values of Q_T/Q_{mean} in the group (regardless of stations) were noted and considered to be the four highest in the sample of size *M* (*M* is the number of station years in the group). They were associated with the expected order statistics $E\ (y_{(1,M)})$, $E\ (y_{(2,M)})$,... Four such values were taken from each group. These new values from all groups were also averaged over intervals of *y* such as *4.0* to *4.5*, *4.5* to *5.0* and the average was plotted. When all the averages had been plotted a smooth curve was drawn through the lower points and extended up to about $y=5.5$ with the guidance of the additional points obtained from the group maxima. The resulting curve was taken as the estimate of the population curve.

GEV-PWMs procedure

This regionalization technique attempts to improve the graphical procedure adopted by the NERC (1975) in the UK flood studies. This involves the following steps:

i. The flow data at each site are arranged in ascending order, *i.e.* $x_1 \le x_2 \le \ldots\ldots\ldots\ldots \le x_n$.
ii. The probability-weighted moments (PWMs) at each site are estimated.
iii. The PWMs are standardized for each site by their sample mean.
iv. The regional PWM estimators are calculated as the weighted average of the standardized at-site PWMs.
v. The regional GEV parameters are estimated using the regional PWM estimators.
vi. For each site, the at-site quantile is estimated by multiplying the at-site mean with the corresponding quantile obtained from the regional GEV distribution.

Hall (1992) illustrated the application of this technique in a case study for the islands of Java and Sumatra in Indonesia. The PWM approach to constructing the regional growth curve was contrasted with that employed in the Flood Design Manual for Java and Sumatra (FDMJS, 1983), which followed the FSR-NERC procedure. The results have shown that for some stations the PWM estimates appear to provide better representation of the recorded data than the FDMJS but for some others neither the PWM nor FDMJS procedure captures the form of the at-site distribution.

Methodology

Regional flood frequency analysis was done by fitting the GEV distribution to the observed flow data of 92 stations of Java and Sumatra using the PWMs method. This follows the assumption that the 92 catchments belong to a single homogeneous region.

The regionalization procedure is as follows:

i. The observed annual peak flows for each site *i* are arranged in ascending order, that is, $x_1 \le x_2 \le x_3 \ldots \le x_j \ldots \le x_n$

ii. The probability weighted moments b_{ri} $(r=0,1,2,\ldots)$ for each site *i* are calculated using the following equations:

$$b_{ri} = \frac{1}{n}\sum_{j=1}^{n} p_j^{\,r} x_j \qquad r=0,1,2,\ldots. \tag{6}$$

$$p_j = \frac{j-0.35}{n} \tag{7}$$

iii. The PWMs are standardized for each site by their sample mean $Q_{mean}=b_{0i}$, *i.e.*

$$T_{0i} = 1 \ ; \qquad T_{1i} = \frac{b_{1i}}{b_{0i}}; \qquad T_{2i} = \frac{b_{2i}}{b_{0i}} \tag{8}$$

iv. The regional PWMs R_0, R_1 and R_2 are calculated as weighted average of the standardized at-site PWM using the following equations:

$$R_0 = \ ; \ R_1 = \frac{\sum_{i=1}^{N} T_{1i} n_i}{\sum_{i=1}^{N} n_i}; \ R_2 = \frac{\sum_{i=1}^{N} T_{2i} n_i}{\sum_{i=1}^{N} n_i} \tag{9}$$

Where, n_i is the sample size for each site *i* and *N* is the number of stations.

v. The parameters of the regional GEV distribution are estimated using the estimators of regional probability-weighted moments with the help of following equations:

$$\hat{k}_R = 7.8590 c_R + 2.9554 c_R^{\,2} \tag{10}$$

Where, $$c_R = \frac{2R_1 - R_0}{3R_2 - R_0} - \frac{\log 2}{\log 3} \tag{11}$$

and k_R is the regional shape parameter

For given value of k_R, the value of a_R and b_R can be obtained from the following equations as:

$$a_R = \frac{(2R_1 - R_0)k_R}{\Gamma(1+k_R)(1-2^{-k_R})} \tag{12}$$

$$b_R = R_0 + \frac{a[\Gamma(1+k_R)-1]}{k_R} \tag{13}$$

Where, a_R *and* b_R are the estimators of scale and location parameter respectively and Γ is the Gamma function.

vi. The dimensionless regional flow quantiles q_T corresponding to an arbitrary return period T can be obtained using the equations given below:

$$q_T = b_R + \frac{a_R}{k_R}\left[1-\left(-ln\left(1-\frac{1}{T}\right)\right)^{k_R}\right] \qquad \text{if } k_R \# 0$$

$$q_T = b_R + a_R\left[1-\left(-ln\left(1-\frac{1}{T}\right)\right)\right] \qquad \text{if } k_R=0 \tag{14}$$

The values of the parameters of the regional GEV distribution obtained for the combined Java and Sumatra data set were: shape parameter, k_R=-0.0342; location parameter, b_R=0.8381; and scale parameter, a_R=0.2646. Applying the Z-test given by Hosking et al. (1985), using the null hypothesis H_0: k_R=0, the PWM growth curve could reasonably be assumed to come from the EVI distribution with location parameter, b_R=0.8422 and scale parameter, a_R= 0.2733. Therefore, the regional frequency curve (also called the growth curve) can be expressed as follows:

$$q_T = 0.8422 + 0.2733\left[-ln\left(-ln\left(1-\frac{1}{T}\right)\right)\right] \tag{15}$$

The regional frequency curve thus obtained can be scaled by the mean annual maximum flood, Q_{mean} of a catchment to estimate the T-year flood, Q_T of that catchment, i.e.

$$Q_T = Q_{mean} \times q_T \tag{16}$$

In case of non-gauged sites, *MAF* can be obtained from the regression equation based on catchment characteristics. For the present study, the regression equation obtained by stepwise regression procedure was employed which is as follows:

$$MAF = 10^{-3.887}\, AREA^{0.78}\, AAR^{1.241}(1+PLTN)^{-1.769}(1+LAKE)^{-2.282} \tag{17}$$

Where, *AREA* is the catchment area in km^2, *AAR* is the catchment average annual rainfall in mm, *PLTN* is the plantation index, and *LAKE* is the lake index.

Equation (17) was compared with the regression equation including POT (peaks-over-a-threshold) series developed by Flood Design Manual For Java and Sumatra (FDMJS, 1983) which is of the form given below:

$$MAF = 8.00 \times 10^{-6}\, AREA^{V}\, APBAR^{2.445}\, SIMS^{0.117}(1+LAKE)^{-0.85} \tag{18}$$

Where, $V = 1.02 - 0.0275 \log_{10} ARE$ (19)

APBAR is the mean annual maximum catchment 1-day rainfall and *SIMS* is the simple slope.

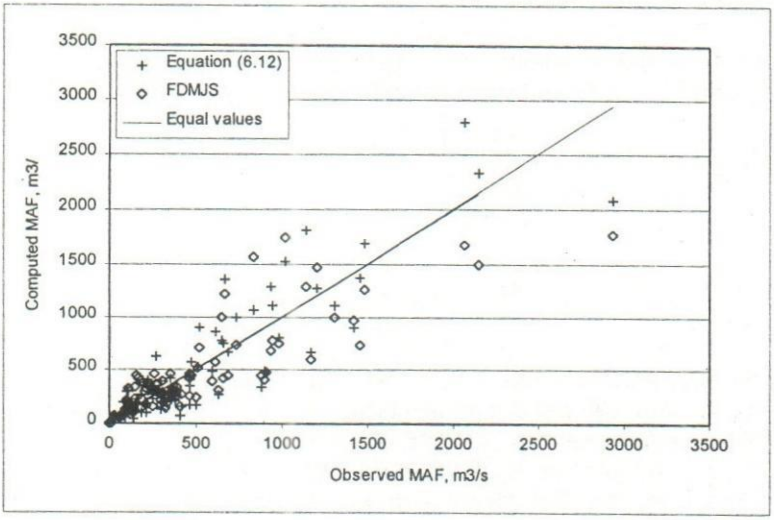
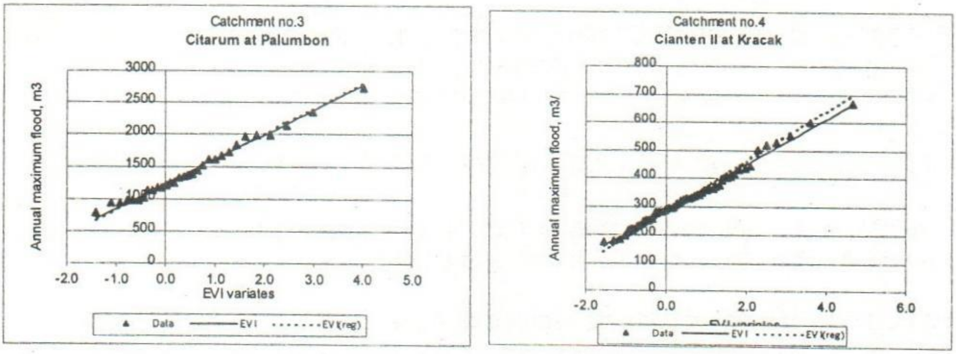


Figure 1 Plot of computed versus observed *MAF* for 92 stationsof Java and Sumatra

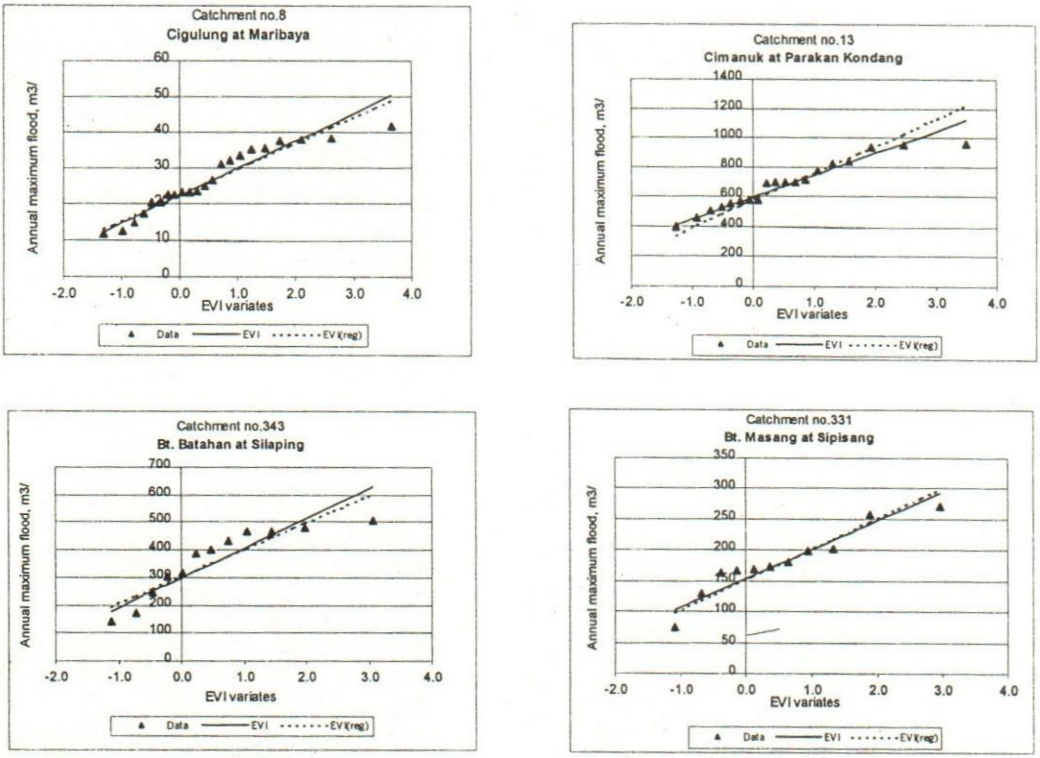
Figure 1 shows plots of the regression *MAF* from Equation (17) and the regression *MAF* from Equation (18) against the 'observed' *MAF* from the FDMJS (1983) for the 92 catchments of Java and Sumatra. The result from the regression model using Equation (17) was found to be superior in terms of RMSE to that provided by the FDMJS regression model; the RMSE between computed and observed values were 242.13 m^3/s and 267.67 m^3/s respectively. Sample plots of the regional EVI distribution scaled by the at-site mean together with at-site EVI for some stations are presented in **Figure 2**.

Figures 2 Sample plots of the regional EVI scaled by the at-site mean together with at site EVI



Figures 2 Sample plots of the regional EVI scaled by the at-site mean together with at site EVI

Figure 2 (cont.)



Figures 2 Sample plots of the regional EVI scaled by the at-site mean together with at site EVI

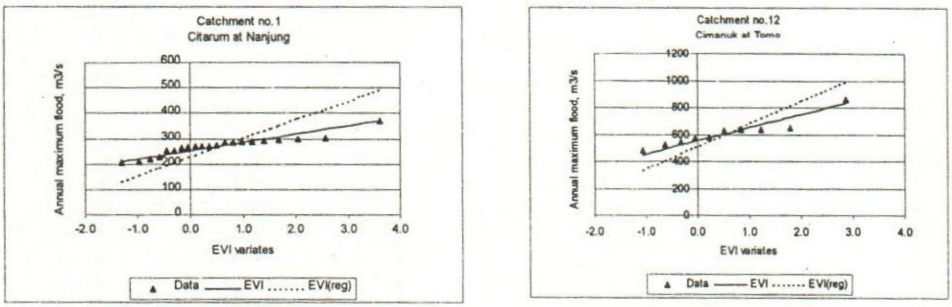
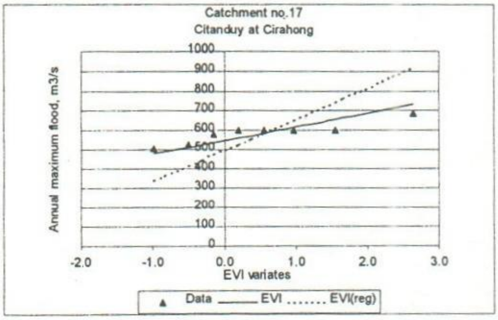
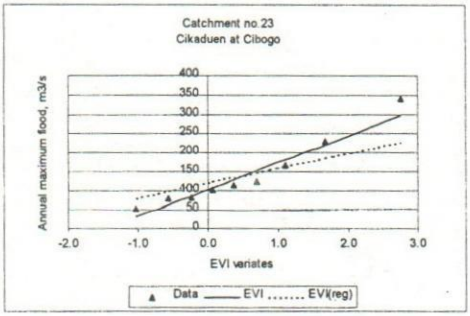
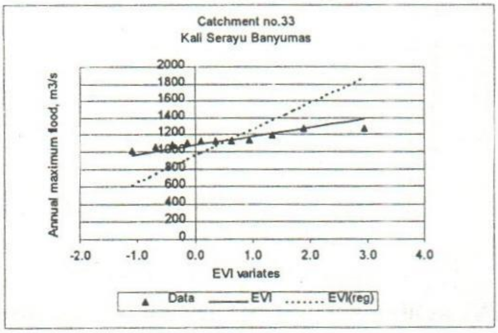


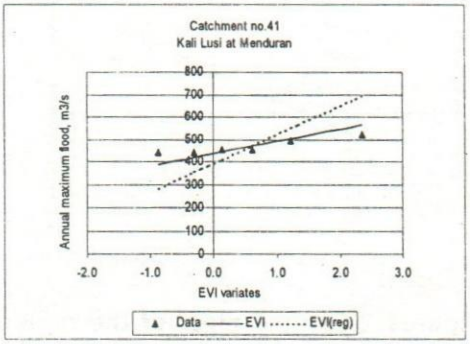
Figure 3 Obvious departures from the data points observed in 14 stations.

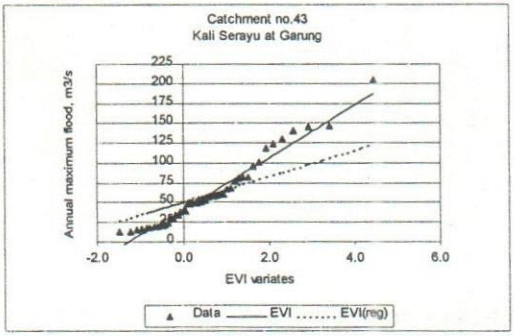
Figure 3 (cont.)











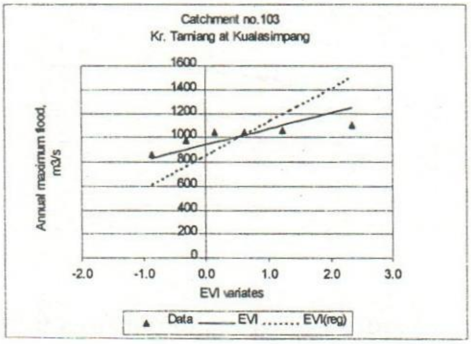
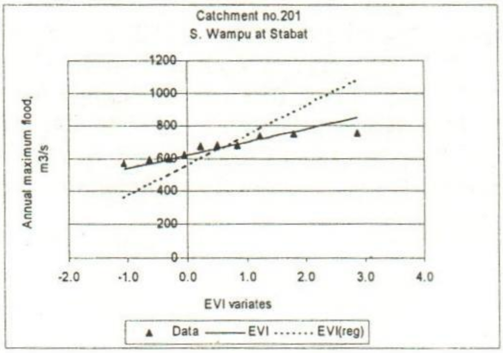
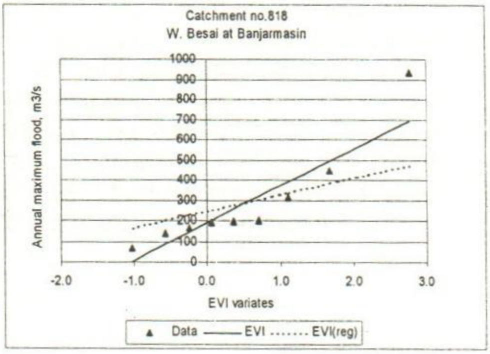
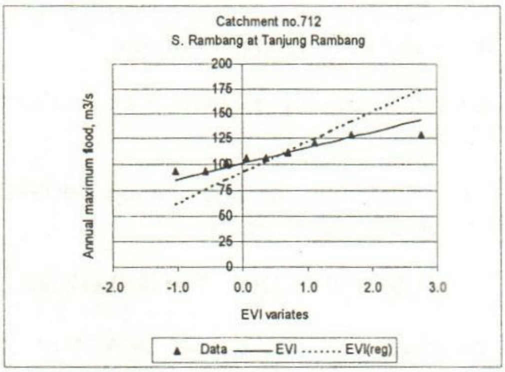
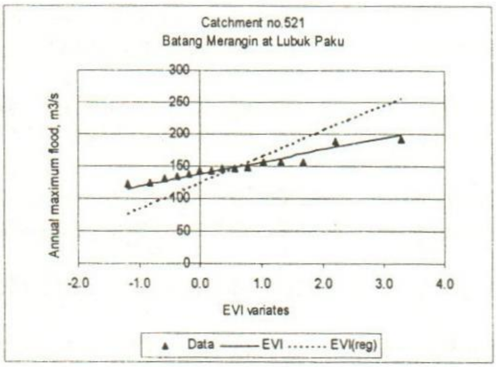


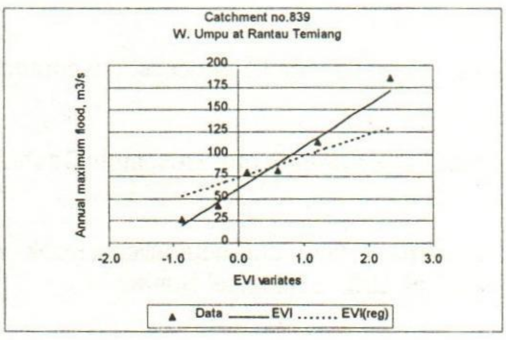
Figure 3 (cont.)

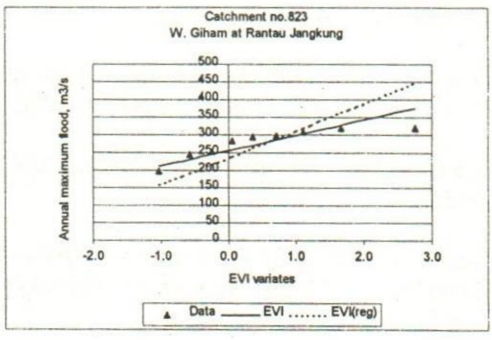












Conclusions

From the plots for the rest of the stations, it can be observed that the scaled regional EVI curve captures the data points of the majority of the 92 stations. Obvious departure from the data points can be noticed on 14 stations. These are stations 1, 12, 17, 23, 33, 41 & 43 in Java and stations 103, 201, 521, 712, 818, 823 & 839 in Sumatra. The results were not surprising because stations 1, 17, 103, 201 & 823 are dominantly EVIII type and stations 23 & 818 are dominantly EVII type; thus these stations can be considered as outliers. For the other 7 stations (12, 33, 41, 43, 521, 712 & 839), the possible explanation may be that these stations belong to different flood regions, which have not been detected by the present methodology.

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INVESTIGATION OF BANK EROSION BY THE JAMUNA RIVER AT NAKALIA PECHAKOLA

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Abstract

The bank erosion is a common problem along the major rivers in Bangladesh. It sometimes causes serious problem as happened in the Nakalia Pechakola, Pabna, due to the Jamuna river right bank erosion for the last couple of years. This paper presents the results and findings of a research project undertaken at River Research Institute (RRI) in the last financial year 2001-2002. The aim of this study is to develop the understanding of the nature and extent of bank erosion and its mitigation measures. It appears from the study that the reason behind this erosion might be due to the fact of highly erodible bank and bed material, very dynamic morphology of the river which changes the flow direction frequently and finally the natural shifting tendency of the Jamuna towards the west. The maximum right bank erosion at Nakalia Pechakola is estimated about 1.4 km towards the west within the years 1969-2001.The river has a widening tendency at the study area, which is seen from the historical spot image i.e. satellite image analysis and cross sectional data analysis. The mean bed level has an increasing trend, indicating the river bed is rising and on the other hand the section is widening.

Introduction

The bank erosion along the bank of the Jamuna river is a regular phenomena. In recent years, the erosion of Nakalia Pechakola on the right bank of the Jamuna in Pabna district became more significant. Each year the river is devouring embankments, cropping land, permanent installations, shopping center, school, and mosque. Communications are disrupted and large numbers of people are homeless. This is situated at the downstream of the confluence of Hurashagar and Jamuna in the Bera Upazila of Pabna district. The erosion of this area is a big issue for the last few years. In the meantime, due to bank erosion, the villages such as Char Nakalia, Maldahpara, Pechakola, Mohanganj, Nayanpur, Sarasia, Char Sarasia, Ballartop, Ganapatdia, Khanpura, Natakhola etc. have already been engulfed by the Jamuna.

The source of the Jamuna River is at Tibet on the northern slope of the Himalayas. It drains an area of about 550,000 km^2, is extended over China, Bhutan, India, and Bangladesh. The total length is about 2,740 km before meeting with the Ganges River at Aricha. The maximum and annual average discharge is recorded 100,000 m^3/s (in 1988) and 20,000 m^3/s respectively. The bank full discharge is about 48,000 m^3/s. The slope of the river within Bangladesh decreases in the downstream direction and is $8.5*10^{-5}$ at the upstream end, and $6.5*10^{-5}$ near the confluence with the Ganges. The bed material sizes also decrease from the upstream towards the downstream part varies from 0.22

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mm to 0.16 mm. The Jamuna is one of the largest braided rivers in the world. It is very dynamic and also complex in nature. The present study, undertaken by the River Research Institute, might help to develop the understanding of the characteristics of the river and to apply structural measures to mitigate the severe bank erosion in the problem area.

Literature Review

Several studies have been carried out on the Jamuna river bank erosion and bank protection of which FAP, CADP and EGIS are important.

FAP Study

FAP- 24 (1996) studied the river bank erosion of the Jamuna river in details and mentioned that it is a complex process in which many factors are involved. The important factors are flow, sediment transport, channel geometry and bed topography, vegetation and ground water level and their variation in time and space and bank and bed material properties.

Bank erosion generally occurs along outer bends due to the presence of secondary flow currents in a meandering river. This secondary flow makes surface layer and bottom layer flow in a different direction. As the maximum sediment concentration is in the bottom layer, in a river bend the direction of the average flow follows the bend axis, while the average sediment transport directed towards the inner bend, thus liable for the formation of point bar. In a meandering river the river bend is generally well defined. But in a braided river, the bend is formed as a part of one of the anabranches. In such a river there is a continuous change in distribution of the water and sediment in between the different anabranches. Which makes a river bend unsTable and less defined than in the case of a meandering river. The flow structure in such a bend is also very complicated.

The bank erosion rates vary along the bend and the maximum bank erosion occurred at the downstream end of the bend. As a result, the maximum erosion rate migrated 1.5 km in downstream direction from August 1993 to November 1995. The maximum bank erosion rate per year at the river bend downstream of Bahadurabad is estimated about 800 m, which can be classified as an extreme event of bank erosion (Klaassen and Masselink' 1992 after FAP-24). The bank erosion estimated by the CADP was compared with the EGIS study. This is shown in Table 1.1 and 1.2.

Table 1.1: Bank Erosion Rates along Pabna Irrigation and Rural Development Programe (Source: CADP'2000)

Location (KM)	1992-97	1997-99	1999-00	1975-00	1992-00	Max
94	40	63	111	17	55	111
93	58	0	111	19	50	111
92	126	28	105	37	99	126
91	115	179	99	48	129	179
90	94	253	110	49	136	253
89	51	185	198	40	103	198
Mean	81	117	122	35	95	

Table 1.2: Bankline Migration at Mathura Reach of Jamuna River (Source: EGIS, 1988 after CADP)

Bank Erosion (m/year)	1973-98	1973-80	1980-89	1989-96	1996-98
Maximum	47	230	49	126	213
Minimum	-4	-101	-31	-22	-393
Median	20	17	14	21	0
Std. Dev.	20	74	19	37	118
Mean	24	40	13	32	-6

CADP Study

Halcrow group limited under the Command Area Development Project (CADP) conducted the Pre-Feasibility and Feasibility study for the mitigation of Jamuna-Meghna river erosion. In the following section, the bank erosion of the Jamuna river at Pabna Irrigation and Rural Development Program (PIRDP) is discussed briefly.

Pre-Feasibility stage

CADP (2000) conducted pre-feasibility study to assess the pattern of bank erosion on the Jamuna river and identifies alternative measures for reducing the erosion threat at the same area. It is mentioned that the bank erosion rates have averaged around 120 m/year to 250 m/year. The riverbank has shifted westward about 1.6 km at some locations since 1975. It is identified that the erosion is severe at PIRDP, particularly in the reach between Nakalia and the Kaitala pump station.

Three long term options assessed in pre- feasibility stage are:

- Embankment retirement
- Bank protection by spurs or revetment
- Channel relocation

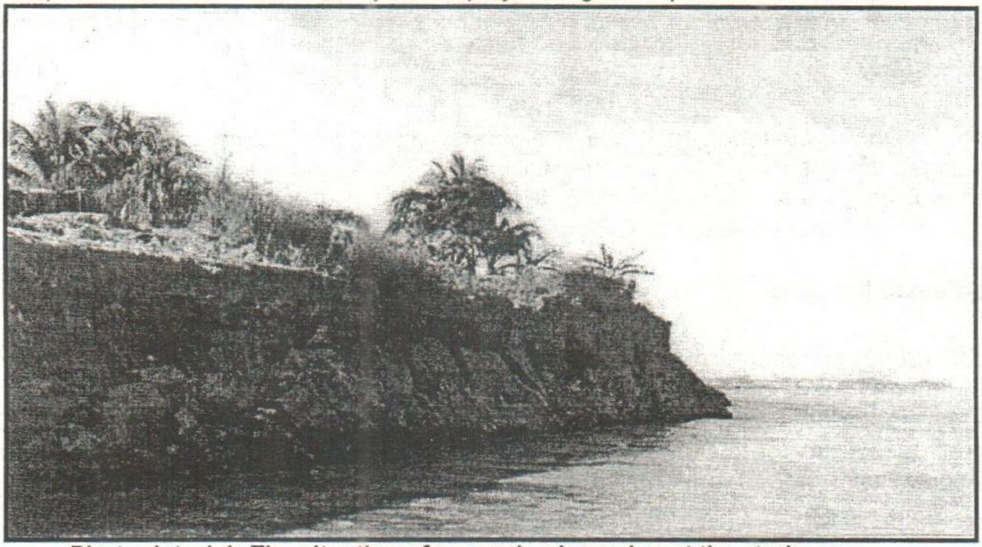
Feasibility Stage

After in depth study in CADP the following measures were suggested in the Feasibility level. (i) erosion mitigation option 1-revetment, (ii) erosion mitigation option 2-solid spur and (iii) embankment retirement option.

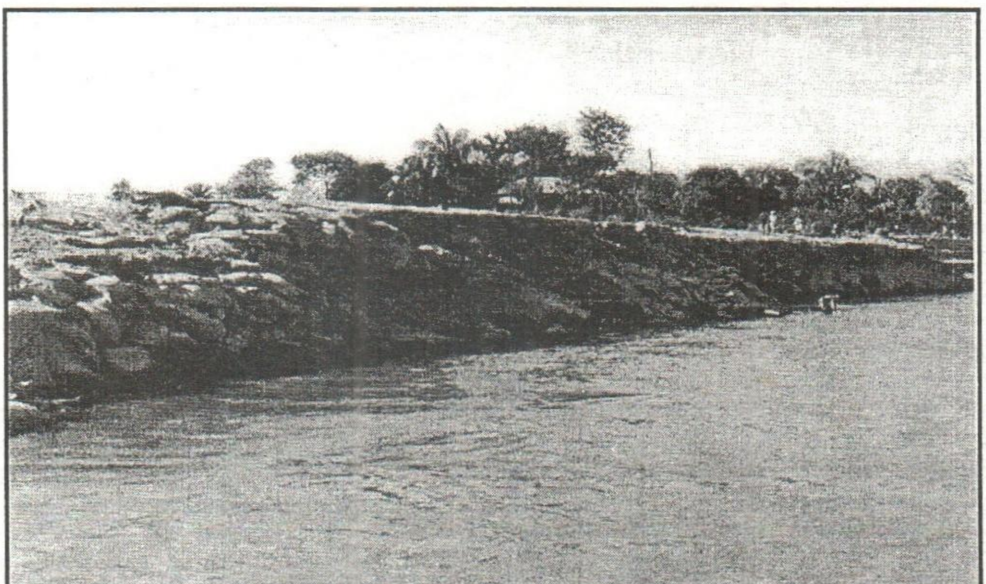
Geo-bags were selected for the design of the erosion mitigation options. The erosion mitigation works are based on an "Adaptive Approach" that is sympathetic to the river morphology and provides flexibility to adapt to morphological changes. The main works comprise of bank stabilization revetments incorporating geo-bag aprons, and Vetivar grass turfing at the upstream and downstream terminations. The new revetment lengths is proposed to 2.7 km at PIRDP. The alternative option of low spurs constructed with geo-bags would only be viable on the basis of cost consideration if there were a rise in the riverbed levels. The existing revetment at both locations constructed by BWDB in 2001 and 2002 will also be strengthened. Also necessary structural improvements required for the flood embankments in these reaches will be undertaken. The works that may be required in the future have also been identified based on current information.

Nature of bank erosion in the field

Field visit is made at the study area for the visualization of the nature and extent of bank erosion. Photoplate 1.1 shows the severe bank erosion and photoplate 1.2 the construction works, which was done on emergency basis to save the specific area and the part of it is damaged within one year flood. So, from the partial damage of the structure, the important point can be stressed here that in addition to the other hydro-morphological study, it is wise to investigate the effectiveness of the proposed hydraulic structure by proper physical modelling which in fact represents the nature. Otherwise, cost optimization and sustainability of the project might be questionable.



Photoplate 1.1: The situation of severe bank erosion at the study area.



Photoplate 1.2: Bank erosion after facing one year flood at the site where emergency

Methodology

The study is carried out based on the past studies, hydro-morphological data analysis and the observation from the field visit. The following data were analyzed in the study:

- Satellite image
- Cross section

To analyze the historical bankline movement, the satellite images have been used for the year 1973, 1985, 1996, 1998 and 2001. The images are taken mainly during the dry period. The scale of these images is 1:100000. The 24°0'N line is considered as a reference line to compare the bank erosion in various years, just downstream of the present study area. A series of parallel lines are drawn upto 9 km upstream of the reference line and the distance between one line to other is 1 km. To identify these lines the base line is marked as 0 km and at the u/s it is increased by 1 km which is indicated in the Table 1.3. The cross sectional data are analyzed to find out the cross sectional area, width of the river, average depth with reference to the bank elevation etc. The bank elevation at different sections is selected after analyzing the cross section of different years. Mean bed level is calculated by subtracting the average depth of the section from the bank level.

Results and discussion

Different types of data that are analyzed in this study are satellite image and cross sectional data. The study was conducted on the right bank of the Jamuna river including some km upstream and downstream of the confluence between Hurashagar and Jamuna covering the Nakalia Bazar in the Pabna district. Kaitala pump station is very close to the downstream to the study area. The study area is shown in Figure 1.1.

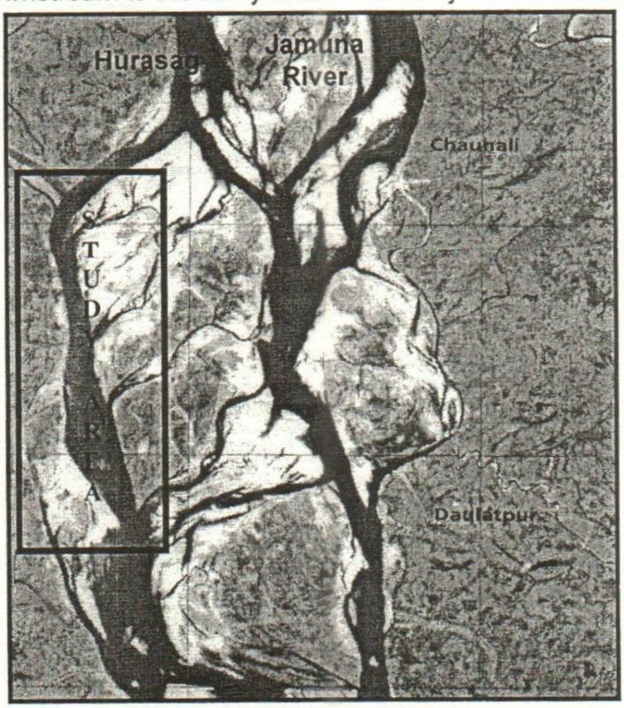


Figure 1.1 Study area

The spot image and cross sectional data were analyzed and discussed in the following. To analyze the historical bankline movement, the satellite images have been used for the year 1973, 1985, 1996, 1998 and 2001. The bank erosion in different years can be seen in Figure 1.2 & 1.3.

The amount of bank erosion at various selected locations is estimated from the Figure 1.2 and shown in Table 1.3. It can be seen from the Figure 1.2 that the bank is shifting in almost every time towards the west. That is still there is a tendency of further bank erosion.

Table 1.3: Bank erosion during different intervals compared to 2001

Distance in km, u/s of the reference line (24°0'N)	Bank erosion in m for the Period			
	2001-1998	2001-1996	2001-1985	2001-1973
1	280	400	350	750
2	250	580	1030	1110
3	100	550	1200	1320
4	120	420	880	1100
5	0	80	350	600
6	0	200	220	350

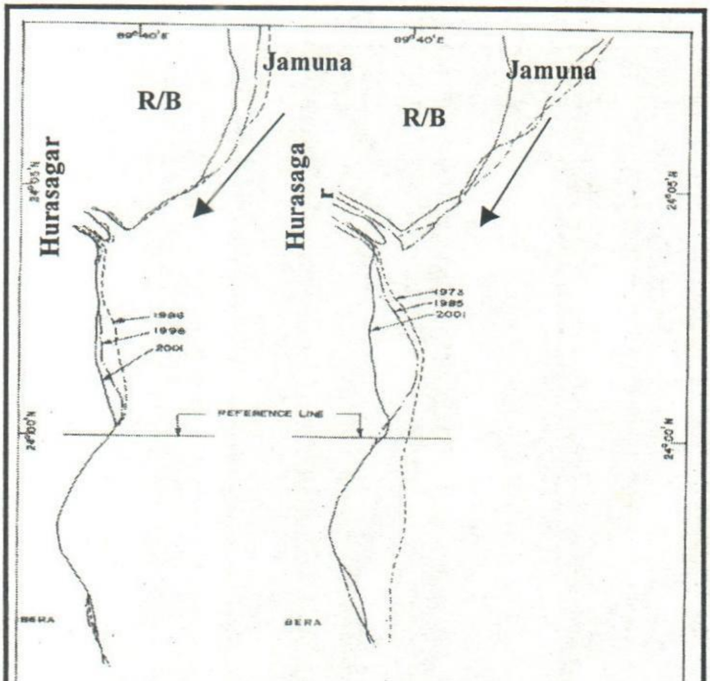


Figure 1.2: Bank shifting in different years at Nakalia Pechakola

Variation of width and men bed level

The width of the river in the study reach is analyzed from the traced bankline of spot images. It can be seen from the Figure 1.3 that the average width of the river is increasing.

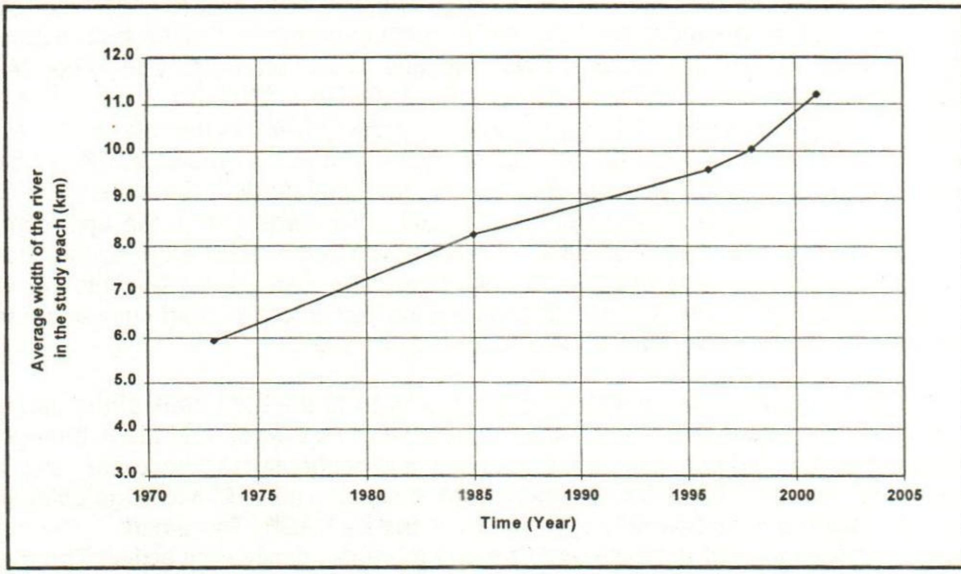


Figure 1.3: The variation of average width in the study area

BWDB cross sectional data analysis shows that the cross sectional area is changing frequently. The width of the river is also showing an increasing trend.It is observed from the mean bed level analysis that the bed level is rising gradually in the study area and specifically within 1981-1999. This phenomenon indicates the sequel widening tendency of the river in general.

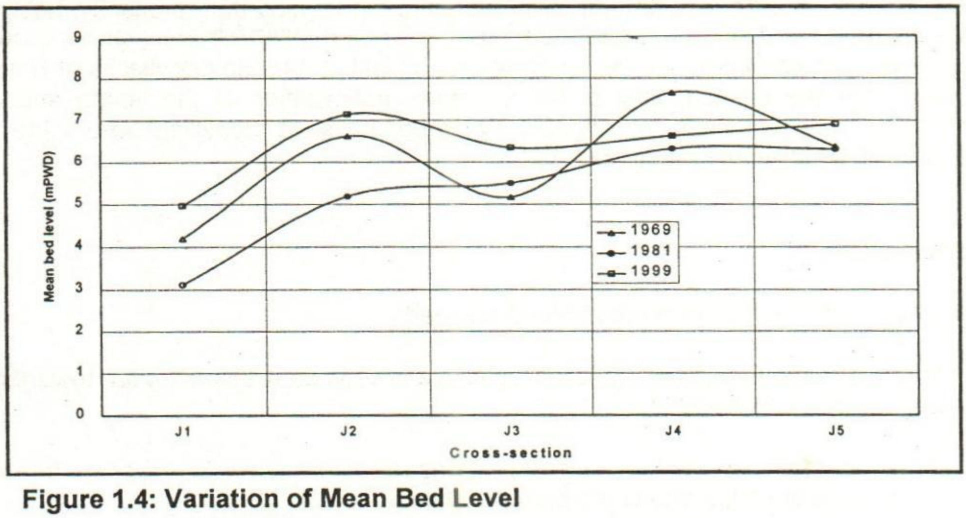


Figure 1.4: Variation of Mean Bed Level

Interpretation of the results

It appears from the spot image analysis as well as the cross sectional data that the right bank has an eroding trend since 1975. The amount of bank erosion varied between 21m to 1.4 km. The maximum amount of bank erosion estimated is about 1.4 km, which is happened between 1973-2001. The EGIS (EGIS' 2002) studied that the bend averaged bank erosion rate at meander bends in the major ana-branches ranges from 0 to 500 meter per year. In extreme cases it even exceeds 1000 meter per year. The CADP estimated the maximum right bank shifting 1.6 km (CADP, 2000). So, it can be stated here that the right bank has a general tendency of shifting towards the west in the study area. It can also be found from the satellite images that there is erosion at the left bank of the Hurashagar near the confluence of Jamuna and further downstream of the confluence. Bank erosion is prominent at the right bank starting from the upstream of Hurashagar confluence and it continues further downstream even after Kaitola Pump House. The reasons behind this may be highly erodible bank and bed material, very dynamic morphology of the river, which changes the flow direction frequently and finally the natural shifting tendency of the Jamuna towards the west.

The satellite image analysis indicates the bank erosion at the right bank of the Jamuna river in the study area is prominent. So to reduce the bank erosion and the sufferings of the dwellers in the study area around the bank it is essential to take necessary steps to mitigate bank erosion. The total maximum bank erosion within 1973-2001 is estimated about 1.4 km which is comparable to the study made by CADP. The nature of the cross section indicates the river is highly unstable and morphologically very active. The mean bed level shows that the river bed is silting up and which causes consequently the widening of the river. The discharge variation of the Jamuna river is very high and the river morphology respond quickly to adopt with the new situation. It is very difficult to combat erosion of the river like Jamuna by non-structural measures, as its morphology is very active and carries huge amount of sediment from the upstream as well as from the adjacent bank erosion. So, It is wise to apply the structural measures to mitigate the bank erosion in such critical situation. The type of structure to be used depends on the detail hydro-morphological study, which was not possible within the limited scope of the present study. The effort made in this paper can only help to develop the understanding of the problem and to recommend for further study. It is good that, presently, BWDB is undertaking the project to mitigate the bank erosion at the PIRDP area, where in-depth study has been carried out by the Halcrow Group Limited. But the drawbacks of Halcrow study is that the sustainability of the structure, optimization of the length and cost optimization has not been investigated by proper physical modelling, where the real behavior of the structure can be tested.

Conclusion

From this study the following conclusions are made:

- The right bank erosion at Nakalia Pechakola is estimated about 1.4 km towards the west between 1973-2001.
- The river has a widening tendency at the study area, which is seen from the historical spot image and cross sectional data.

- The mean bed level has an increasing trend, indicating the river bed is rising and on the other hand the section is widening.
- The effective structural measures are needed to combat erosion at the Nakalia-Pechakola area. The type of structure to be constructed is better to select after detail physical model investigation.

List Of Abbreviations

BWDB	Bangladesh Water Development Board
CADP	Command Area Development Project
EGIS	Environmental and Geographic Information System
FAP	Flood Action Plan
IWM	Institute of Water Modeling
JMREP	Jamuna Meghna River Erosion Mitigation Project
PIRDP	Pabna Irrigation and Rural Development Project
PMU	Project Management Unit
R/B	Right Bank
RRI	River Research Institute

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ARSENIC INVESTIGATION OF DRINKING WATER IN SOME AREAS OF FARIDPUR DISTRICT

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Abstract

Water is very essential for existence of life. Now this water has become cause of death due to contamination of arsenic. The arsenic contamination in drinking water is now one of the most vital problem in Bangladesh. High concentration of arsenic in ground water has created serious problem in recent years. Arsenic has contaminated in the ground water of 61 districts out of 64 districts of Bangladesh. At present about 35 million of people of Bangladesh are at high risk due to consumption of arsenic contaminated water. Under this situation RRI took up a research project on Arsenic Investigation of Drinking Water in some Areas of Faridpur District. The objective of the study was to detect level of arsenic contamination in drinking water of tube wells. In this research study a large number of tube well water samples were collected from within the Faridpur pourashava and out side the pourashava areas adjacent to RRI to detect the level of arsenic concentration. The researchers also collected informative data about the arsenic contamination in tube well water of different Upazillas and Unions of Faridpur from DPHE and NGO, World Vision, Faridpur. The researchers analyzed the collected samples & available information and reported the findings with some recommendations. This paper will provide comprehensive information on the status of arsenic contamination in tube well water of some areas of Faridpur district. The paper also puts some suggestions and recommendations for mitigation of arsenic contamination.

Introduction

In Bangladesh, water supply for drinking purpose is almost entirely dependent on groundwater extracted from shallow tube wells. Now this water has become poisonous by arsenic contamination. In Bangladesh the contamination of arsenic in ground water was detected at first by DPHE at Chapai Nawabgang in 1993. The earlier discovery of arsenic in ground water in west Bengal followed the diagnosis of arsenic poisoning in 1978. The location of affected villages in West Bengal led many conclusions that the border districts of Bangladesh must also be contaminated to some degree. Various surveys during 1995 and 1996 confirmed these opinions, but more surprisingly found that contamination extended across large parts of the country, perhaps affecting even more people than in India (British Geological Survey, 1998).

During the last three decade a large number of tube wells had been drilled though out the country by the Department of Public Health Engineering and other Non Government Organizations with the financial aid of UNICEF and many other donors to save the people from the water borne diseases. Now this drinking water has become poisonous

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by arsenic contamination. High concentration of arsenic in ground water has created serious problem in recent years. Specialists and Scientists suspect that the arsenic contamination in drinking water in Bangladesh is probably the largest mass poisoning case in the world now. An International Workshop for mitigation of arsenic problem was arranged by the ministry of the local government, Rural Development and Co-operative during the period of 14-16 January, 2002. In this workshop Specialists and Scientists opined that arsenic has contaminated in the ground water of 61 districts out of 64 districts of Bangladesh (Source : The Daily Ittefaq, 14th January, 2002). They also suspect that at present about 35 million of people of Bangladesh are at high risk due to consumption of arsenic contaminated water. The specialists also mentioned in the workshop that the number of tube-wells drilled by Government and Private sectors are about 13 and 80 lacs respectively. They opined that for mitigation of arsenic problem it is required firstly, to test the water of all the tube-wells and detect arsenic level in each of the tube-wells. Secondly, to supply drinking water from alternative sources where arsenic level in drinking water is beyond permissible limit. Thirdly, to develop techniques for removal of arsenic from drinking water and lastly to find out the sources of arsenic contamination in drinking water.

Faridpur is one of the worst arsenic affected areas of Bangladesh. Under this circumstance RRI had under taken a research project by its own fund to find out the level of arsenic in drinking water in some areas of Faridpur. RRI also collected information about arsenic contamination in ground water of various Upazillas and Unions of Faridpur from DPHE and World Vision respectively. In this research study attempts were made to develop awareness among the people about the adverse effect of drinking arsenic contaminated water and put some suggestions & recommendations for mitigation of arsenic.

Literature review

Anwar (2000), it is mentioned that in Faridpur and many places in Bangladesh deaths due to arsenic poisoning has been reported, Shahidul Islam of Harshava, Faridpur recently died at the age of 25 due to drinking water containing arsenic more than 1.7 mg/l. The whole family and others like Usha Rani Sutrodhor, Momta Begum, Mubarak Hossain of Dhakin Tapakhola and many other places are now waiting painful end of their life without getting any help or advise or knowing the reasons of this misery. Also reported that Arsenic contaminated ground water has been mainly detected in the upper and main aquifer and possibly deep aquifer is also contaminated in areas where it is hydraulically connected with the over lying main aquifer . The high risk arsenic polluted districts of Bangladesh are Chapai Nawabgonj, Rajshahi, Kushtia, Meherpur, Chuadanga, Satkhira, Bagerhat, Faridpur, Pabna, Jessore etc. **Sobhan, et.al, (1999),** discussed that arsenic and its compounds are mobile in the environment. Weathering of rocks convert arsenic sulphides to arsenic trioxide which enters the arsenic cycle as dust or by dissolution in rain, river or ground water. Human exposure of arsenic occurs primarily from air, food and water. They also mentioned that the areas vulnerable to arsenic contamination are the Ganges floodplain, the tidal region, the coastal plain and the Meghna flood plain. Some specialists opine that the arsenic problem in ground water particularly in southern region of Bangladesh is probably due to arsenic rich deltaic deposition and hydraulic connection with contaminated region of West Bengal province of India. But from the analysis of initial studies, it is clear that the main areas of contamination are the Ganges dependent areas of Bangladesh.

According to British Geological Survey (1998), arsenic in ground waters from major aquifers have shown that the occurrence can be widespread depending on regional geological and environmental conditions. Arsenic concentrations significantly higher than drinking water standards have been found in ground water of large parts of Argentina, Chile, Taiwan, Inner Mongolia, Mexico, Western USA, Thailand, Japan, India and Bangladesh. Many of the worst cases of arsenic contamination arise in reducing ground water where anaerobic condition favors mobilization of arsenic substantially as arsenite. Examples of reducing aquifers include Taiwan and Inner Mongolia as well as Bangladesh and west Bengal.

Anwar (2000), reported that Studies of several affected populations, Particularly those in Argentina and chile (0.25 million people exposed for several decades), have revealed a close association between skin cancer and arsenic exposure. Similarly the incidence of Black foot disease in Taiwan, manifested by gangrene of extremities, was found to increase in a dose-dependent manner with arsenic. He also mentioned that mass out break of arsenic poisoning occurred in young children in summer of 1955 in Japan.

Muslim (2002), it is reported that Water of in total of 53500 nos. of tube wells have been tested and found the arsenic contamination in the ground water of 61 district out of 64 district of Bangladesh. But level of contamination of ground water is different in different areas. 268 Upazilla of 61 district have been found arsenic contamination in ground water. It has been found that 28% of the tube well have arsenic concentration exceeding the acceptable limit (0.05 mg/l) in Bangladesh. This contamination is mainly found in shallow tube well (STW). Arsenic contamination in deep tube well is (0.7%) as much less than STW.

Paribeshpatra (2002), it is stated that South and south - east districts are most affected arsenic contaminated areas of Bangladesh.

Badruzzaman et., al. (1998), reported that water in about 61.1% of the tube wells in the north-eastern zone have arsenic concentration exceeding the acceptable limit set by the WHO (0.01 mg / l) for drinking water and about 33.2% exceed the Bangladesh standard value (0.05 mg / l).

Observer Magazine (2001), it is reported that About 80 million people in Bangladesh and the Indian state of west Bengal are now exposed to arsenic poisoning as most of the people still have been drinking arsenic contaminated water at levels between 5 and 100 times the WHO guide line value.

Study area

The study areas for arsenic investigation were Dangi, Kaita Kalibari under Nagar Kanda thana, Gerdha, Harukandi, Khabaspur, Alipur, Mahmudpur, Char Kamlapur, Ambikapur, Char Tepakhola, Gandia, Mallikpur, Goal Chamot, Kamlapur under sadar thana and Baksipur under Modhukhali thana of Faridpur district. Information data about arsenic contamination in tube well water of Char Bhadrason, Alfadanda, Modhukhali, Boalmari, Nagarkanda, Sadarpur, Bhanga and Faridpur Sadar Upazilla were collected from DPHE and the same data of Eshan Gopalpur, Aliabad, Char Madhabdia, koizuri, Gerda, Dikrir Char and Ambikapur unions were collected from the NGO World Vision.

Methodology

Method of data collection

Each sample collected in plastic container, was labeled separately with a unique identification number. A register khatha was maintained with the same unique number of each sample with the information of the owner of the tube well, village, mouza, union and thana. It also included the information of the collection date, depth of the tube well, description of disease and previously arsenic test was done or not.

Testing procedure

The samples were tested using MERCK Arsenic Test Kit no. 1.17926.0001. Zinc and sulfuric acid were added to compounds of arsenic (iii) and arsenic (v), arsenic hydried is liberated, which in turn reacts with mercury (ii) bromide contained in the reaction zone of the analytical test strip to form yellow–brown mixed arsenic mercury halogenides. The concentration of arsenic (iii) and arsenic (v) were measured by visual comparison of the reaction zone of the analytical test strip with the fields of a colour scale. Measuring range/colour scale graduation were **0.00, 0.01, 0.025, 0.05, 0.1, 0.5 mg / l $As^{3+/5+}$.**

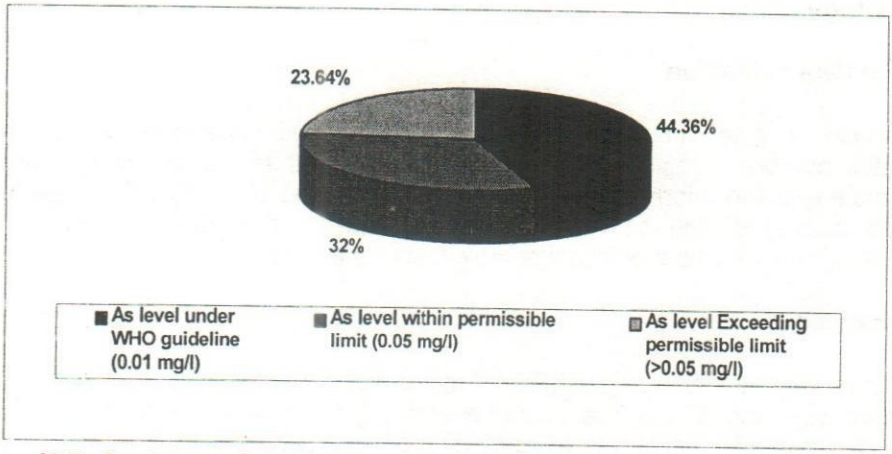
Removed 1 analytical test strip and immediately reclosed the tube. With the reaction zone first inserted the test strip about half way through the slot in the stopper of the reaction vessel. By means of the syringe, transferred 10 ml of the solution to be tested to the reaction vessel and added 2 measuring spoonfuls of reagent As–1(zinc). Rapidly added 10 drops of reagent As-2 (sulfuric acid), immediately closed the reaction vessel with the stopper and swirled gently. The sample solution was not come in to contact with the test strip. Leave to stand for 30 minute, gently swirled two or three times. Removed the test strip, briefly dip into water, shaked off excess liquid and determined with which colour field on the label the colour of the reaction zone coincides most exactly. Read off the corresponding concentration value in mg / l $As^{3+/5+}$. If an exact colour match could not be achieved, estimated an intermediate value.

Results and discussion

A total of 305 nos.tube well water sample were tested of Faridpur district. Among these 275 samples were under Faridpur Pourashava and 30 samples were outside the Pourashava. Information collected from DPHE and World Vision were 26908 and 11852 nos. of tube well samples respectively.

Arsenic concentration under Faridpur Pourashava area

Arsenic concentration in the tube wells under Faridpur pourashava area has been shown in the form of pie diagram in **Fig. 1.** It is observed from the figure that the arsenic concentration of about 44.36% of the tube wells were under WHO guideline (0.01 mg/l), about 32% of tube wells within the permissible limit of Bangladesh standard (0.05 mg/l) and about 23.64% of tube wells exceeding permissible limit (> 0.05 mg/l).



N.B. As means Arsenic

Fig. 1 Arsenic concentration under Faridpur Pourashava

Arsenic concentration outside the Pourashava area

Arsenic concentration outside the pourashava area has been presented in the form of pie diagram in **Fig. 2**. It is apparent that the arsenic concentration of about 13.33% of the tube wells were under WHO guideline (0.01 mg/l), about 6.67% within permissible limit of Bangladesh standard (0.05 mg/l) and about 80% of tube wells exceeding permissible limit (> 0.05 mg/l).

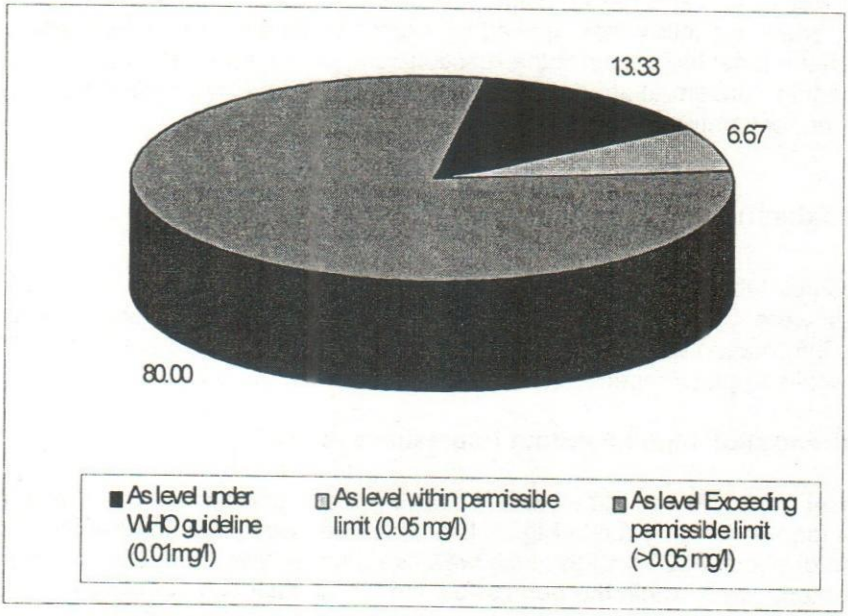


Fig. 2 Arsenic concentration out side the Pourashava area

It is seen from **Figs. 1 & 2** that out side the pourashava area, the percentage of tube wells exceeding permissible limit is much higher than within Pourashava area. In our investigation only 30 nos. of tube well water samples out side pourashava area were tested. If large no. of samples were collected & tested from out side the pourashava area then percentage of tube wells exceeding permissible limit might probably decrease.

Arsenic contamination in Faridpur as a function of depth

A depth-wise variation of arsenic concentration in the tube wells is shown in **Fig. 3**. In general arsenic contamination decreases with the increase of depth. BGS (1998) stated that at Tungipara, Faridpur and Manikganj arsenic concentrations were found to increase with depth. Our investigation also indicates that the arsenic contamination increases with the increase of depth. Depth bellow 50 ft a total of 41 no. of tube well samples were tested. Out of tested samples 92.68% of tube wells were arsenic level under WHO guideline (0.01 mg/l), 4.88% within permissible limit (0.05 mg/l) and 2.44% exceeding permissible limit (>0.05 mg/l). Depth between 50 to 100 ft a total of 94 no. of tube well samples were tested. Out of tested samples 40.43% of tube wells were arsenic level under WHO guideline (0.01 mg/l), 34.04% within permissible limit (0.05 mg/l) and 25.53% exceeding permissible limit (>0.05 mg/l). Depth between 101to 150 ft a total of 51 no. of tube well samples were tested. Out of tested samples 25.49% of tube wells were arsenic level under WHO guideline (0.01 mg/l), 39.22% within permissible limit (0.05 mg/l) and 35.29% exceeding permissible limit (>0.05 mg/l). Depth between 151to 250 ft total of 51 no. of tube well samples were tested. Out of tested samples0% of tube wells were arsenic level under WHO guideline (0.01 mg/l), 40% within permissible limit (0.05 mg/l) and 60% exceeding permissible limit (>0.05 mg/l).

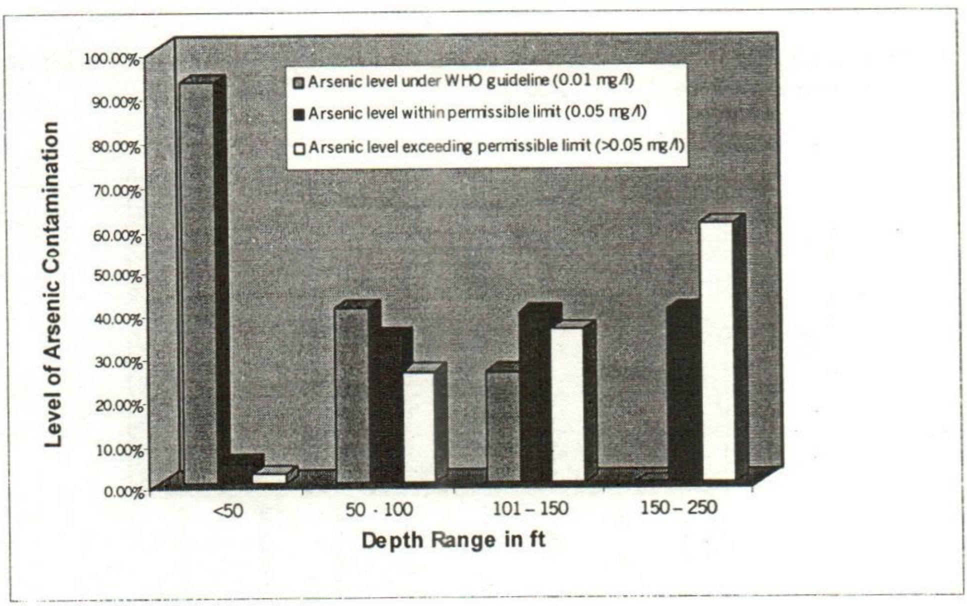


Fig. 3 Arsenic Contamination of tube well water in Faridpur as a Function of Depth

Arsenic contamination in tube well water of various Upazilla under Faridpur district

Arsenic contaminated tube well water of various upazillas under Faridpur district has been graphically presented in Fig. 4(a) & 4 (b). The figures show that total no. of tube well, no. of tube well of arsenic test, no. of arsenic contaminated tube well and percentage of arsenic contaminated tube well of various Upazillas in Faridpur district. From the graphs it is seen that nos. and percentages of arsenic contaminated tube wells are highest at Bhanga and lowest at Madhukhali.

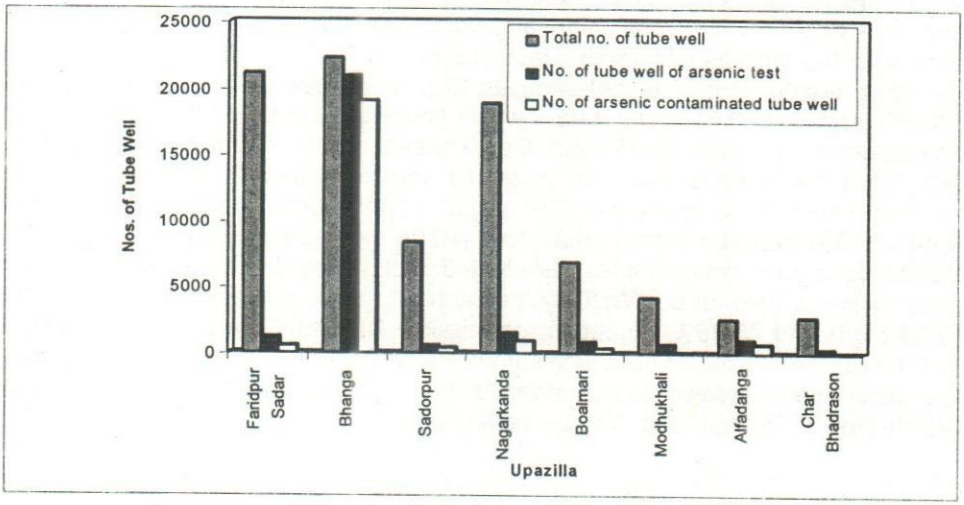


Fig. 4a Arsenic contaminated tube wells of various Upazillas under Faridpur district (Source: DPHE, 2002)

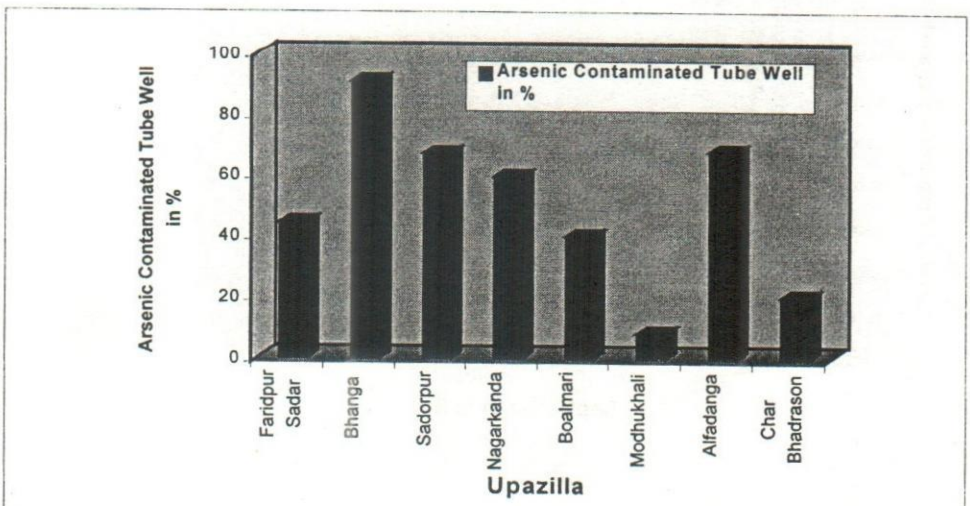


Fig. 4b Percentages of arsenic contaminated tube wells of various Upazillas under Faridpur district (Source: DPHE, 2002)

Drinking & non drinking tube well water of various Unions in Faridpur

Drinking, non-drinking, tested total no. of tube well water and percentages of Arsenic contaminated tube wells of various Unions in Faridpur has been presented graphically in **Fig. 5a** and **Fig. 5b** respectively. It is observed from the graph that the nos. of non drinking (red) and percentages of arsenic contaminated tube wells is highest in Koizuri Union and lowest at Eshan Gopalpur Union.

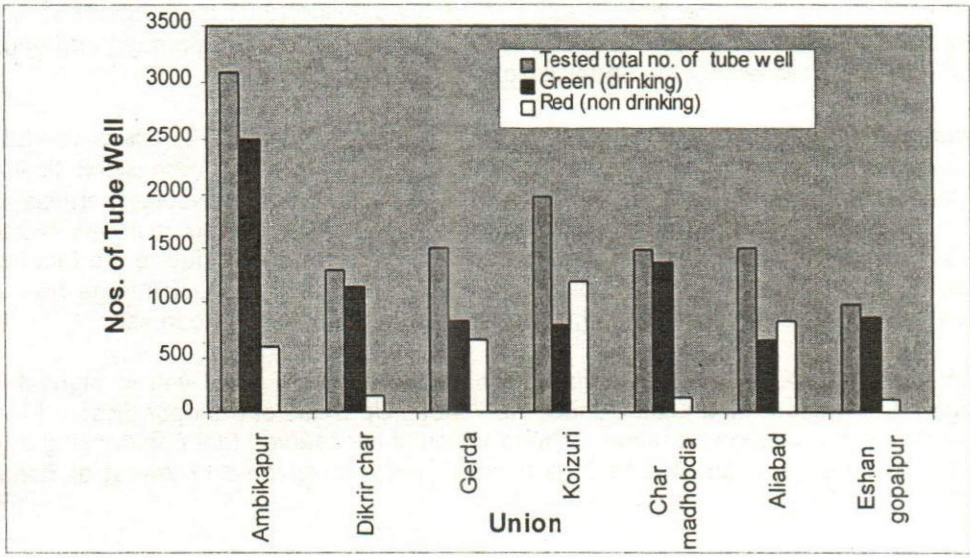


Fig. 5a Drinking & non drinking tube well water of various Unions in Faridpur (Source: World Vision, 2002)

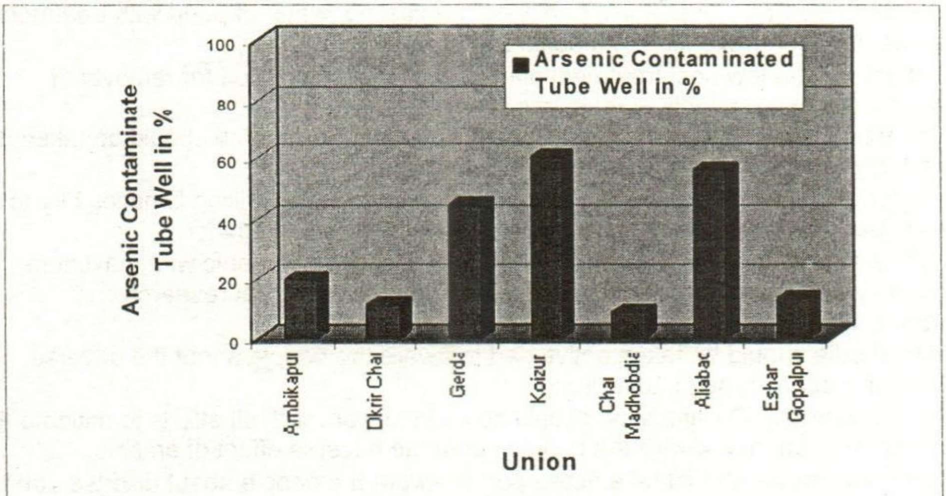


Fig. 5b Percentages of non drinking (arsenic contaminated) tube well of various Unions in Faridpur (Source: World Vision, 2002)

Conclusion and recommendations

Conclusion

From the results based on the outcome of the study, it is observed that most of the areas of Faridpur are arsenic affected. Pourashava area has been less affected than the outside of the Pourashava. Among the area under Pourashava, Char Kamlapur has the highest percentage of tube wells contaminated with arsenic. Arsenic contamination increases with the increase of depth. Under Pourashava area, the arsenic concentration of about 44.36% of the tube wells are under WHO guideline (0.01 mg/l), about 32% of tube wells are within the permissible limit of Bangladesh standard (0.05 mg/l) and about 23.64% of tube wells exceeding permissible limit (> 0.05 mg/l).

Out side the Pourashava area, the arsenic concentration of about 13.33% of the tube wells are under WHO guideline (0.01 mg/l), about 6.67% are within permissible limit of Bangladesh standard (0.05 mg/l) and about 80% of tube wells exceeding permissible limit (> 0.05 mg/l). The percentages of tube wells exceeding permissible limit (>0.05 mg/l) is very much higher out side the Pourashava area. It may be due to the fact that only a few nos. of samples were collected and tested of the areas. If a large nos. of samples were collected and tested, the percentages might probably decrease.

Information obtained from DPHE that arsenic contamination tube well is highest in Bhanga and lowest in Modhukhali under the various Upazillas of Faridpur district. From the arsenic contamination statistics of World Vision, it is observed that non drinking tube well due to contamination of arsenic is highest in Koizuri union and lowest at Eshan Gopalpur.

Recommendations

The following recommendations can be put for remedy of arsenic:

- Alternative source of water like rain water harvesting, water of pond with treatment can be used for prevention of arsenic contamination.
- In affected area low cost treatment plant should be constructed for removal of arsenic in order to supply arsenic free water.
- In arsenic affected areas if possible piped water supply scheme should be taken up to supply treated water from surface water source.
- In order to identify the sources of arsenic in affected areas drilling is necessary to investigate the geochemistry and hydrology of the affected aquifer.
- Approprite and sophisticated equipments for detection of arsenic with maximum accuracy should be supplied from the Government to different research organizations those are engaged in arsenic related problem.
- News media should be taken active part to aware the people about the adverse effect of arsenic through advertising.
- Non Government Organization should come in forward with all efforts to mitigate the arsenic problem and aware the people about the adverse effect of arsenic.
- Multi mass media should take active part to aware the people about disease due to arsenicosis through Radio, TV and news paper.

- Arsenic affected people should consume huge amount of vegetables and nutricious food (like meat, fish, egg, milk, pulse etc.) to prevent arsenic related disease.
- For detail investigation of arsenic situation of Faridpur, it is required to test the water samples of all the tube wells of this area.

Acknowledgement

The authors are grateful to the Department of Public Health Engineering (DPHE) and World Vision, Faridpur for supplying valuable information regarding arsenic contamination in tube well water of various Upazillas and Unions of Faridpur district. They are also thankful to those who helped in preparing this paper.

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FLOOD FREQUENCY ANALYSIS USING AT-SITE DATA: A CASE STUDY

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Abstract

The magnitude of flood quantiles for a given return period is usually estimated by flood frequency distribution to be fitted to the flood data. In this paper, the parameters of Generalized Extreme Value (scale, location & shape parameter) and that of Extreme Value Type I distributions (scale & location parameter) have been determined by utilizing the estimated probability-weighted moments. Then the Z-test proposed by Hosking et al. (1985) has been applied to the shape parameter of Generalized Extreme Value distribution to find out whether or not Extreme Value Type I distribution can be used rather than Generalized Extreme Value distribution. The result shows that all the 92 stations except 8 stations have observations lying outside the 95% confidence limits. There are 5 stations where Extreme Value Type III distribution is more suitable than Extreme Value Type I distribution and 1 station where Extreme Value Type II distribution is more suitable than Extreme Value Type I distribution.

Introduction

Flood frequency analysis is a technique for estimation of flood magnitude at a predetermined return period. For that purpose, a standard flood frequency distribution is to be fitted to the chosen data and the value of the parameters such as scale, location & shape of that distribution is to be determined from the moments.

The choice of a suitable standard frequency distribution is often controversial, but the Generalized Extreme Value (GEV) distribution has obtained widespread acceptance (Hall & Minns, 1998). In the present study GEV and Extreme Value Type I (EVI) distributions have been considered. Next to determine is if EVI distribution can be used rather than the GEV distribution or not. The Z-test proposed by Hosking et al. (1985) has been applied to the shape parameter of GEV distribution.

For the 92 catchments of Java and Sumatra, the parameters of the GEV distribution as well as those of the EVI distribution have been estimated using the method of probability-weighted moments (PWMs). This method is chosen because the small-sample properties of PWM estimators of parameters and quantiles for distributions of the GEV family are generally superior to those of the method of moments and the method of maximum likelihood (Hosking et al., 1985).

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Methodology

Checking of annual maximum flood data

Hydrological time series may exhibit trends referred to as inconsistencies or non-homogeneities. Inconsistencies result from changes in the amount of systematic errors associated with recording of data, such as those arising from changes in instrumentation or observational practices. Non-homogeneity is defined as a change in the statistics of the data set, which are caused by natural or man-made change. Split record tests on variances and means are applied to detect the presence of inconsistencies or non-homogeneities. These tests are referred to as the *F*-test for stability of the variance and *t*-test for stability of the mean. These two tests can be reinforced by a third test, Spearman's rank correlation test, for indicating absence of trends. All three tests determine the presence or absence of 'absolute' consistency or homogeneity, as they are performed on an individual data series without comparison with other series (de Laat, 1998)

The annual maximum flood data that will be used in the flood frequency analysis requires data to be stationary, consistent and homogeneous. To determine whether or not the data have satisfied these criteria, the following tests recommended by Dahmen and Hall (1990) have been applied.

Trend test using Spearman's rank correlation method

The Spearman's rank correlation coefficient R_{sp} can be expressed as:

$$R_{sp} = 1 - \frac{6\sum_{i=1}^{n} D_i^2}{n(n^2-1)} \tag{1}$$

Where, D_i is the difference between the position in the data set as observed and the rank of the same data set when arranged in ascending order, and n represents the number of observations.

For testing the null hypothesis H_o: $R_{sp}=0$ against an alternative hypothesis H_1: $R_{sp} \neq 0$, the test statistic t used in this study can be defined as:

$$t = R_{sp}\left[\frac{n-2}{1-R_{sp}^2}\right]^{\frac{1}{2}} \tag{2}$$

Where, the test statistic t has a Student's t-distribution with $v=n-2$ degrees of freedom. Applying a two-tailed test with a level of significance of 5% the null hypothesis H_o is accepted, indicating that there is no trend, if

$$t(v,2.5\%) < t < (v,97.5\%) \tag{3}$$

F-test for the stability of variance and *t*-test for the stability of mean

The steps for testing the stability of variance and mean of the time series are as follows:

1. Split the time series into two sub-sets at the point where a (supposed) change in the characteristics of data occurs. If there is no obvious change in behavior, either of the following rules can be applied: (1) divide the time series into two equally non-overlapping sub-sets; or (2) divide the time series into two-third and one-third non-overlapping sub-sets; or (3) divide the time series into one-third and two-third non-overlapping sub-sets.

2. Test if the variances, S_1^2 and S_2^2, of the two sub-sets are not significantly different from each other. The test statistic used for testing the null hypothesis, H_o: $S_1^2=S_2^2$ against an alternative hypothesis H_1: $S_1^2 \neq S_2^2$ can be expressed as:

$$F_t = \frac{S_1^{\ 2}}{S_2^{\ 2}} \tag{4}$$

Applying a two-tailed test with a significance level of 5%, the null hypothesis is accepted *i.e.* the variance of the time series is stable, if

$$F(v_1, v_2, 2.5\%) < F_t < F(v_1, v_2, 97.5\%) \tag{5}$$

Where, the test statistic F_t has an *F*-distribution with $v_1=n_1-1$ and $v_2=n_2-1$ degrees of freedom in the numerator and denominator respectively. n_1 and n_2 are the length of records of the two sub-sets.

3. Test if the means, $\overline{X}_1$ and $\overline{X}_2$, of the two sub-sets are not significantly different from each other. To do this the null hypothesis $H_o : \overline{X}_1 - \overline{X}_2 = 0$ is used against an alternative hypothesis $H_1 : \overline{X}_1 - \overline{X}_2 \neq 0$ and the test statistic used for testing the hypothesis can be defined as:

$$t = \frac{\overline{X}_1 - \overline{X}_2}{S_p \sqrt{\frac{1}{n_1} + \frac{1}{n_2}}} \tag{6}$$

Where, S_p is the pooled estimate of the variance and is calculated by the following formula:

$$S_p = \left[\frac{(n_1 - 1)S_1^{\ 2} + (n_2 - 1)S_2^{\ 2}}{n_1 + n_2 - 2} \right]^{\frac{1}{2}} \tag{7}$$

Where, n_1 and n_2 are the length of records of the two sub-sets. Applying a two-tailed test with a significance level of 5%, the null hypothesis is not rejected, which means that the mean of the data set is stable, if

$$t(v, 2.5\%) < t < t(v, 97.5\%) \tag{8}$$

Where, the test statistic t has a Student's *t*-distribution with $v=n_1+n_2-2$ degrees of freedom.

The tests were implemented using data screening (DATSCR) computer program developed by Dahmen & Hall (1990). Of the 92 stations, 9 stations failed in the Spearman's rank correlation test (Trend-test), 14 stations failed in the test for the stability of variance (*F*-test), and 6 stations failed in the test for the stability of mean (*t*-test). The summary of the test results for the stations that failed in one of the tests is given in **Table 1**. Strictly speaking, these stations should be discarded but since there are limited stations available, they were still included in the present analysis.

Table 1 Name of the stations that failed in at least one of the tests

Station no.	Name of station & location	Name of the tests		
16	Cisanggarung at Cilengkrang		*F*-test	
17	Citanduy at Cirahong		*F*-test	
18	Ciseel at Cilisung		*F*-test	
25	Ciletuh at Cipiring			*t*-test
40	Kali Bogowonto at Bener	Trend-test		*t*-test
41	Kali Lusi at Menduran		*F*-test	
43	Kali Serayu at Garung	Trend-test	*F*-test	*t*-test
46	Bengawan Solo at Bojonegoro		*F*-test	
50	Kali Welang at Porwodadi	Trend-test		*t*-test
314	Batang Kuranji at Gunung Nago		*F*-test	
316	Bt. Anai at Kandang Empat		*F*-test	
343	Bt. Batahan at Silaping	Trend-test		
413	S. Rokan Kiri at Lubuk Bendahara		*F*-test	
422	Bt. Mahat at Sipopay	Trend-test		*t*-test
431	Bt. Agam at Titi		*F*-test	
511	Bt. Hari at Sungai Dareh		*F*-test	
512	Bt. Ule at Lubuk Tapus		*F*-test	
803	W. Sekampung at Pujorahayu	Trend-test		
807	W. Bulok at Jembatan	Trend-test		*t*-test
818	W. Besai at Banjarmasin	Trend-test	*F*-test	
824	W. Umpu at Negeri Batin	Trend-test		
834	W. Rarem at Kota Bumi		*F*-test	

Generalized Extreme Value (GEV) distribution

The GEV distribution was introduced by Jenkinson (1955, see Hall, 1992), and combines into a single expression the three types of limiting distributions for Extreme Values (EV) identified by Fisher and Tippett (1928; see Hall, 1992).

The GEV distribution function may be expressed as:

$$F(x) = exp\left[-\left(1-\frac{k(x-\beta)}{\alpha}\right)^{\frac{1}{k}}\right]; \qquad k\#0$$

$$F(x) = exp\left[-exp\left(-\frac{(x-\beta)}{\alpha}\right)\right]; \qquad k=0 \tag{9}$$

Where, $F(x)$ is the cumulative distribution function of the variable x and α, β, & k are the scale, location & shape parameters of that distribution respectively. The shape parameter, k, determines the appropriate EV distribution, with the Fisher-Tippet Types I, II and III corresponding to $k=0$, $k<0$ and $k>0$, respectively. The range of x is bounded by $\beta+\alpha/k$ from below if $k<0$ and by the same quantity from above if $k>0$. For majority of flood series, the value of k is found to fall within the range $-0.5<k<0.5$ (Hall, 1992).

Estimation of the GEV distribution parameters using the PWMs

Greenwood et al. (1979) introduced the probability-weighted moments, which can be defined as:

$$M_{p,r,s} = E\left[X^p F^r (1-F)^s\right] = \int_0^1 [x(F)]^p F^r (1-F)^s dF \tag{10}$$

Where, p, r and s are real numbers and F is the distribution function of the random variable X. When $r=s=0$ and p is a non-negative integer, Equation (10) reduces to

$$M_{p,0,0} = E\left[X^p\right] = \int_0^1 [x(F)]^p dF \tag{11}$$

which represents the conventional moment of order p about the origin. In estimating the parameters of a distribution of the random variable X, it is convenient to use the moment $M_{1,r,s}$ because the occurrence of only a first power of X would result in a simpler expression than when using the conventional moments. Furthermore, when r and s are integers, $F^r(1-F)^s$ may be expressed as a linear combination of either powers of F or powers of $(1-F)$. Therefore, the estimation of parameters of a distribution can be worked out using the moments either of the forms $M_{1,r,0}$ $(r=0,1,2,\ldots..)$ or $M_{1,0,s}$ $(s=0,1,2,\ldots)$.

Hosking et al. (1985) favored the form $M_{1,r,0}$ and adopted the designation for the probability-weighted moments of order r as β_r. They gave the probability-weighted moments of order r of the GEV distribution for $k \neq 0$ as:

$$\beta_r = \frac{1}{r+1}\left(\beta + \frac{\alpha[1-(r+1)^{-k}\Gamma(1+k)]}{k}\right); \qquad k>-1,\ \Gamma=\text{Gamma function} \tag{12}$$

Therefore,

$$\beta_0 = \beta + \frac{\alpha[1-\Gamma(1+k)]}{k}, \tag{13}$$

$$2\beta_1 - \beta_0 = \frac{\alpha[\Gamma(1+k)(1-2^{-k})]}{k}, \tag{14}$$

and

$$\frac{3\beta_2 - \beta_0}{2\beta_1 - \beta_0} = \frac{1-3^{-k}}{1-2^{-k}} \tag{15}$$

In order to obtain the PWM estimators of β, α and k, Equations (13)-(15) must be solved by replacing the β_r by their sample estimators, b_r, and employing iterative methods. However, because the right-hand side of Equation (15) is almost linear over the range $-0.5<k<0.5$, the following approximate estimators were proposed:

$$k = 7.859c + 2.9554c^2 \tag{16}$$

Where,

$$c = \frac{2b_1 - b_0}{3b_2 - b_0} - \frac{\log 2}{\log 3} \tag{17}$$

b_0, b_1 and b_2 are the estimators of the probability-weighted moments β_0, β_1 and β_2 respectively. b_r may be obtained from the ordered sample *i.e.* $x_1 \le x_2 \le x_3 \ldots \le x_j \ldots \le x_n$ using the following expression:

$$b_r = \frac{1}{n}\sum_{j=1}^{n} p_j^{\,r} x_j \qquad r=0,1,2,\ \ldots \tag{18}$$

Where,

$$p_j = \frac{j-0.35}{n} \tag{19}$$

For given value of k, the estimators a and b of the scale and location parameters respectively, of the GEV distribution may be estimated sequentially from Equations (14) and (13) as:

$$a = \frac{(2b_1 - b_0)k}{\Gamma(1+k)(1-2^{-k})} \tag{20}$$

$$b = b_0 + \frac{a[\Gamma(1+k)-1]}{k} \tag{21}$$

The values of the estimators b_0, b_1 and b_2 of the probability-weighted moments as well as the estimators k, a & b of the shape, scale and location parameters of the GEV distributions for some catchments of Java and Sumatra are presented in **Table 2a** and **2b** respectively.

The shape parameter k of the GEV distribution, which depends on the sample skewness of the observed data, exhibits a high variability. This means that reliable estimate of skew requires very large samples. The average length of the annual maximum flood record used in the present study is only around 11 years, which is relatively short to guarantee reliable estimates of the shape parameters. However, Hosking *et al.* (1985) proposed a test to determine whether or not the shape parameter k in the GEV distribution is not significantly different from zero. The test uses a null hypothesis H_0:

$k=0$ against an alternative hypothesis H_1: $k \neq 0$. The test statistic (Z_t) used for testing the hypothesis can be expressed as:

$$Z_t = k\sqrt{\frac{n}{0.5633}} \tag{22}$$

Where, k is the shape parameter and n is the sample size. The computed value of Z_t is compared to the $Z_{critical}$ (critical value of test statistic) provided by Hosking et al. (1985). When $Z_t<Z_{critical}$, the null hypothesis is accepted and in which case the EVI or Gumbel distribution (a special case of the GEV distribution where $k=0$) can provide a satisfactory description of the observed peak floods.

Table 2a Estimators of the GEV PWMs and the GEV parameters for some catchments of Java

Name of the	**Estimators of the PWMs**			**Estimators of GEV parameters**		
catchment	b_0	b_1	b_2	k	a	b
Citarum at Nanjung	270.14	146.47	101.44	0.290	40.398	256.087
Citarum at Saguling	659.57	388.02	283.13	-0.136	145.825	553.012
Citarum at Palumbon	1446.55	856.25	624.70	-0.069	358.378	1213.408
Cianten II at Kracak	331.26	193.86	140.52	-0.046	77.938	282.594
Cikarang at Cikarang	244.79	144.57	105.42	-0.082	58.964	205.591
Cilamaya at Cipedy	497.25	296.55	213.76	0.275	168.444	436.966
Cigulung at Maribaya	26.61	15.96	11.57	0.204	8.965	22.966
Cikapundung at Maribaya	31.14	17.78	12.73	-0.034	6.167	27.366
Cimanuk at Leuwigoong	294.94	165.62	116.96	0.144	58.847	268.386
Cimanuk at Garut	102.71	60.92	44.43	-0.043	26.480	86.265

Table 2b Estimators of the GEV PWMs and the GEV parameters for some catchments of Sumatra

Name of the	Estimators of the PWMs			Estimators of GEV parameters		
Catchment	b_0	b_1	b_2	k	a	b
Kr. Tamiang at Kualasimpang	1015.03	553.16	381.64	0.581	179.44	981.49
Kr. Jambo Aye at Lhoknibong	931.58	552.38	399.60	0.136	279.17	803.90
S. Wampu at Stabat	664.63	360.49	249.90	0.249	97.65	628.04
S. Belawen at Asam Kumbang	209.00	128.45	94.45	0.104	75.43	172.57
S. Ular at Pulo Tagor	347.39	207.97	151.89	0.018	100.51	291.11
S. Padang at Tebing Tinggi	154.96	90.14	64.77	0.100	39.75	135.63
Bah Bolon at Nagori Bandar	210.80	124.13	89.43	0.163	61.52	183.94
S. Silau at Kisaran Naga	257.71	154.73	114.54	-0.221	58.12	208.08
Bt. Pane at Gunung Tua	873.47	518.02	377.72	-0.038	226.10	734.07
Batang Gadis at Perbangunan	306.07	194.26	144.71	0.102	129.57	243.21

The Z-test was performed for each of the 92 stations of Java and Sumatra. The results are given in **Table 3a** and **3b** for some catchments of Java & Sumatra respectively. The values of Z_t are smaller than $Z_{critical}$ for all catchments. Therefore, the shape parameter within the data set is not significantly different from zero and the EVI or Gumbel distribution can provide a satisfactory description of the data for all the 92 stations of Java and Sumatra.

Table 3a Results of Z-test using H_0:$k=0$ against H_1:$k \neq 0$ for some catchments of Java

Name of catchment	n	K	Z_t	Z_{cr}	Name of catchment	n	k	Z_t	Z_{cr}
Citarum at Nanjung	21	0.290	1.77	3.86	Kawah Ciwicey at Pos A	10	0.223	0.94	3.2
Citarum at Saguling	7	-0.136	-0.48	3.02	Cipadarum at Pos B	12	0.110	0.51	3.32
Citarum at Palumbon	31	-0.069	-0.51	4.24	Ciwidey at Pos C	11	0.215	0.95	3.26
Cianten II at Kracak	58	-0.046	-0.46	4.89	Cisarua at Pos D	13	-0.021	-0.10	3.38
Cikarang at Cikarang	11	-0.082	-0.36	3.26	Cisarua at Pos E	9	0.142	0.57	3.14
Cilamaya at Cipedy	6	0.275	0.90	2.96	Kali Padegolan at Pajengkolan	6	-0.019	-0.06	2.96
Cigulung at Maribaya	22	0.204	1.27	3.92	Kali Serayu at Banyumas	11	0.163	0.72	3.26
Cikapundung at Maribaya	12	-0.034	-0.16	3.32	Kali Progo at Duwet	8	0.448	1.69	3.08
Cimanuk at Leuwigoong	32	0.144	1.08	4.27	Kali Progo at Kranggan II	9	-0.271	-1.08	3.14
Cimanuk at Garut	10	-0.043	-0.18	3.2	Kali Serang at Durungan	10	-0.033	-0.14	3.2

Table 3b Rsults of Z-test using H_0:$k=0$ against H_1:$k \neq 0$ for some catchments of Sumatra

Name of catchment	n	K	Z_t	Z_{cr}	**Name of catchment**	n	k	Z_t	Z_{cr}
Kr. Tamiang at Kualasimpang	6	0.581	1.89	2.96	Bt. Agam at Titi	16	-0.465	-2.48	3.56
Kr. Jambo Aye at Lhoknibong	8	0.136	0.51	3.08	Bt. Hari at Sungai Dareh	6	-0.258	-0.84	2.96
S. Wampu at Stabat	10	0.249	1.05	3.2	Bt. Ule at Lubuk Tapus	6	-0.310	-1.01	2.96
S. Belawen at Asam Kumbang	8	0.104	0.39	3.08	Bt. Tabir at Muara Jernith	8	-0.197	-0.74	3.08
S. Ular at Pulo Tagor	10	0.018	0.07	3.2	Batang Merangin at Lubuk Paku	15	-0.003	-0.01	3.5
S. Padang at Tebing Tinggi	5	0.100	0.30	2.9	Bt. Tembesi at Muara Inum	12	0.321	1.48	3.32
Bah Bolon at Nagori Bandar	10	0.163	0.69	3.2	S. Musi at Despetah	8	0.071	0.27	3.08

Table 3b (Continued)

Name of catchment	n	K	Z_t	Z_{cr}	Name of catchment	n	k	Z_t	Z_{cr}
S. Silau at Kisaran Naga	9	-0.221	-0.88	3.14	Air Komering at Martapura	9	-0.177	-0.71	3.14
Bt. Pane at Gunung Tua	6	-0.038	-0.13	2.96	Air Ogan at Batu Raja	10	-0.326	-1.38	3.2
Batang Gadis at Perbangunan	9	0.102	0.41	3.14	S. Rambang at Tanjung Rambang	9	0.180	0.72	3.14

Estimation of the EVI distribution parameters using PWMs

The EVI distribution is a special case of the GEV distribution. When $k=0$, the GEV distribution becomes an EVI distribution which has only two parameters (scale parameter, α and location parameter, β). The cumulative distribution function may be expressed as:

$$F(x) = exp\left[-exp\left(-\frac{(x-\beta)}{\alpha}\right)\right] \tag{23}$$

Greenwood et al. (1979) obtained the following expressions for scale and location parameters using PWMs:

$$\alpha = \frac{2\beta_1 - \beta_0}{ln 2} \tag{24}$$

$$\beta = \beta_0 - 0.5772\alpha \tag{25}$$

Where, a & b are the estimators of α and β respectively. The values of a & b can be obtained by replacing β_0 by b_0 and β_1 by b_1 in Equations (24) and (25).

$$a = \frac{2b_1 - b_0}{ln 2} \tag{26}$$

$$b = b_0 - 0.5772a \tag{27}$$

The calculated values of the EVI parameters for some catchments of Java and Sumatra are presented in **Table 4a** and **4b** respectively.

Table 4a Estimators of the scale and location parameters of EVI distribution for some catchments of Java

Name of Catchment	Estimators of EVI parameters		Name of catchment	Estimators of EVI parameters	
	a	b		a	b
Citarum at Nanjung	32.897	251.155	Kawah Ciwicey at Pos A	0.126	0.506
Citarum at Saguling	168.033	562.583	Cipadarum at Pos B	2.642	5.045
Citarum at Palumbon	383.701	1225.076	Ciwidey at Pos C	1.665	6.712
Cianten II at Kracak	81.444	284.249	Cisarua at Pos D	1.432	4.288
Cikarang at Cikarang	63.972	207.869	Cisarua at Pos E	1.274	4.380
Cilamaya at Cipedy	138.276	417.434	Kali Padegolan at Pajengkolan	175.338	526.613
Cigulung at Maribaya	7.668	22.183	Kali Serayu at Banyumas	106.897	1073.582

Table 4a (Continued)

Name of Catchment	Estimators of EVI parameters		Name of catchment	Estimators of EVI parameters	
	a	*b*		*a*	*b*
Cimanuk at Leuwigoong	52.375	264.707	Kali Progo at Kranggan II	181.314	270.034
Cimanuk at Garut	27.588	86.789	Kali Serang at Durungan	180.798	297.375

Table 4b Estimators of the scale and location parameters of EVI distribution for some catchments of Sumatra

Name of the Catchment	Estimators of EVI parameters		Name of the Catchment	Estimators of EVI parameters	
	a	*b*		*a*	*b*
Kr. Tamiang at Kualasimpang	131.70	939.02	Bt. Agam at Titi	50.53	79.26
Kr. Jambo Aye at Lhoknibong	249.86	787.36	Bt. Hari at Sungai Dareh	1196.58	2250.43
S. Wampu at Stabat	81.29	617.71	Bt. Ule at Lubuk Tapus	91.36	298.72
S. Belawen at Asam Kumbang	69.11	169.11	Bt. Tabir at Muara Jernith	146.23	419.88
S. Ular at Pulo Tagor	98.90	290.30	Batang Merangin at Lubuk Paku	18.80	136.98
S. Padang at Tebing Tinggi	36.53	133.87	Bt. Tembesi at Muara Inum	306.08	987.70
Bah Bolon at Nagori Bandar	54.04	179.61	S. Musi at Despetah	39.33	194.74
S. Silau at Kisaran Naga	74.67	214.61	Air Komering at Martapura	345.02	1103.20
Bt. Pane at Gunung Tua	234.55	738.08	Air Ogan at Batu Raja	331.32	1217.80
Batang Gadis at Perbangunan	118.94	237.41	S. Rambang at Tanjung Rambang	15.79	100.43

Goodness-of-fit test for EVI distribution

The goodness-of-fit test for the EVI distribution was evaluated using the graphical fitting method. The observed and estimated floods were plotted against the EVI standardized variate, *y* which were obtained using the following expression:

$$y=-ln\ [-ln\ (F_m)] \qquad (28)$$

Where, F_m is the Gringorten (1963; see Fornis, 1998) plotting position and $m=1,2,3,\ldots\ldots n$, are the ranks in ascending order of magnitude. The plotting position F_m has the following form:

$$F_m = \frac{m-0.44}{n+0.12} \qquad (29)$$

The flood estimates corresponding to an arbitrary return period T were obtained using the equation as follows:

$$x(T) = b - a\,ln\left[-ln\left(1-\frac{1}{T}\right)\right] \tag{30}$$

The 95% confidence limits (*CL*) to the quantile estimates were calculated from the following expression:

$$CL(x) = \hat{x} \pm t_{97.5,n-1} SE \tag{31}$$

Where, $t_{97.5,\ n-1}$ is the value of Student's *t*-distribution for a *97.5* percent level of confidence (two-tailed test) and *(n-1)* degrees of freedom. The standard error, *SE* can be obtained using the expression as follows:

$$SE = \frac{C\sigma}{\sqrt{n}} \tag{32}$$

Where, σ is the standard deviation of sample size *n*. For EVI distribution, the value of *C* can be obtained by the expression as follows:

$$C = 0.78\sqrt{1.17 + 0.196\,y + 1.099y^2} \tag{33}$$

Where, *y* is the standardized EVI variate.

Sample plots of the fitted GEV and EVI distributions together with 95% confidence limits for the EVI are shown in **Figure 1**. The stations having observations lying outside the 95% confidence limits are shown in **Figure 2**.

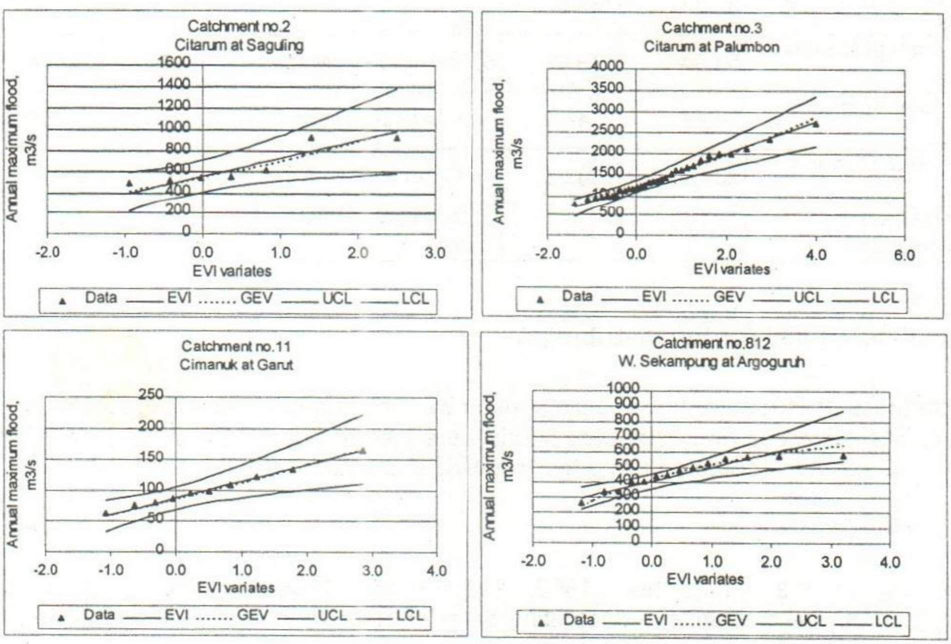

Figure 1 Sample plots of the fitted GEV and EVI distributions together with 95% limits for the EVI distribution.

Figure 1 (Continued)

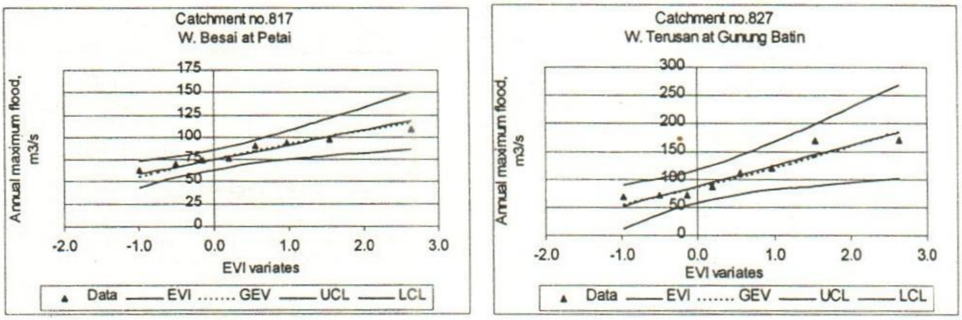

Figure 1 Sample plots of the fitted GEV and EVI distributions together with 95% limits for the EVI distribution.

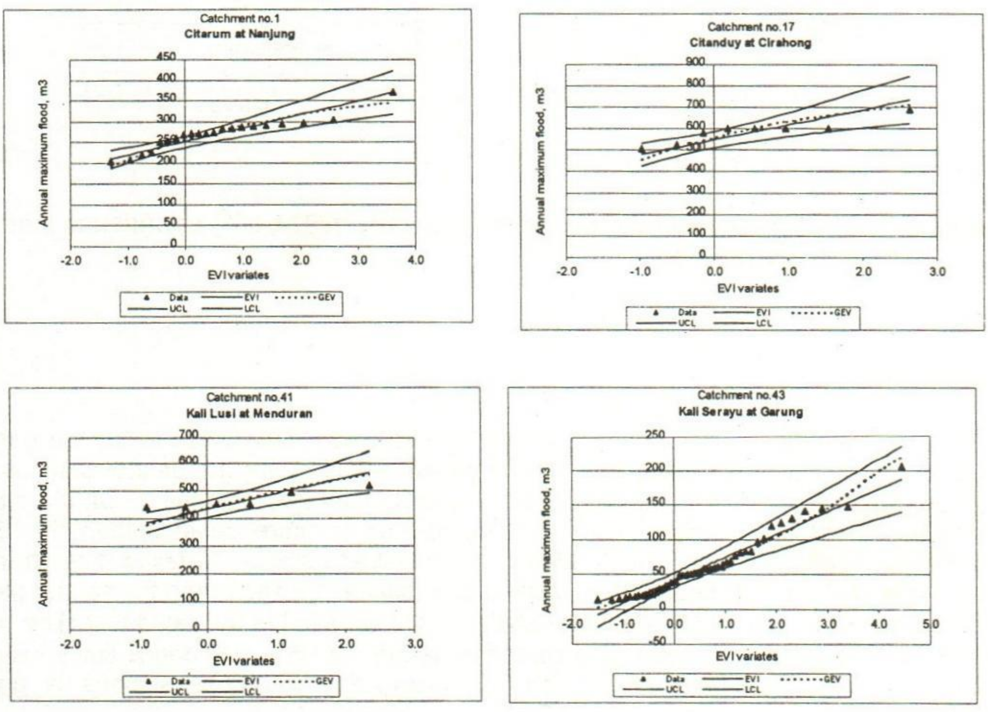

Figure 2 Fitted GEV and EVI distributions lying outside the 95% confidence limits for EVI distribution

Figure 2 (Continued)

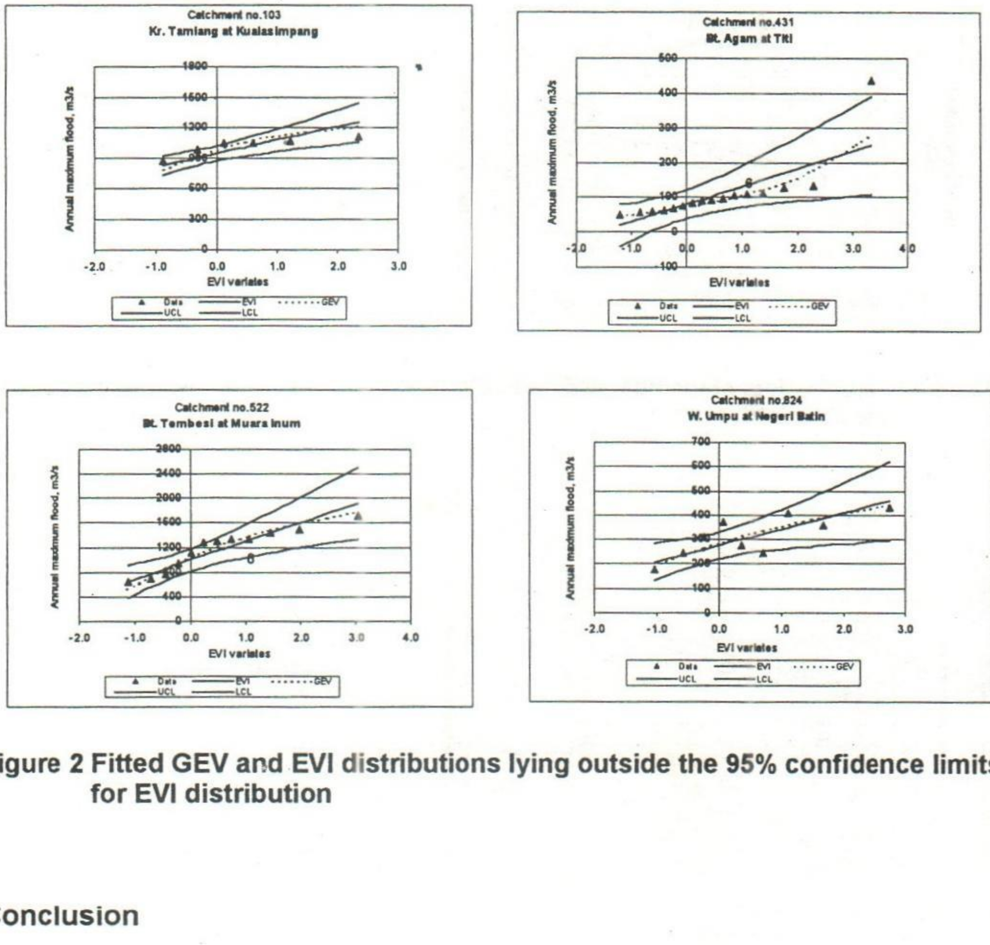

Figure 2 Fitted GEV and EVI distributions lying outside the 95% confidence limits for EVI distribution

Conclusion

There are 8 stations (1, 17, 41 & 43 in Java and 103, 431, 522 & 824 in Sumatra) that have observations lying outside the 95% confidence limits. For the data at stations 1, 17, 103, 522 & 824, EVIII distribution is more suitable than EVI distribution. On the other hand, for the data at station 431, EVII distribution is more suitable than the EVI distribution. Stations 17, 41 & 431 failed in the *F*-test and station 43 failed in the trend-test, *F*-test & *t*-test. The flood data of station 824 that can be seen from the scatter plots are of doubtful nature and it is another station that has failed in the trend-test. The rest of the stations have their flood data contained within the 95% confidence limits, which means that the goodnèss-of-fit of the EVI distribution for these stations is quite acceptable.

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PERFORMANCE OF A SERIES OF RCC SOLID SPURS CONSTRUCTED ON THE FLOOD PLAIN BY PHYSICAL MODELING: A CASE STUDY

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Abstract

In general solid spurs are constructed extended from bankline into the river, which is very much expensive for major rivers of Bangladesh. But recently Bangladesh Water Development Board (BWDB) designs solid spurs in a series to be constructed on the flood plain extended from bankline towards the country side up to flood embankment. These are easier to construct and less costly in the aforesaid rivers. This paper presents the functioning of a series of RCC solid spurs constructed on the flood plain by physical model investigation at River Research Institute (RRI) for the protection of Char Bhadrason area from the erosion of Padma river. From the model study with solid spurs in a series it has been observed that the solid spurs constructed on the flood plain can be used effectively to combat river bank erosion.

Introduction

The study area is located under Char Bhadrason upazila in the district of Faridpur covering a reach of about 10km. Many permanent important installations such as school, college, hat, bazar, ghat, post office, food godown, family planning center, homesteads and many agricultural lands are under threat due to the continuous bank erosion of Padma river. The economy of that area is mainly based on agriculture. A right embankment was constructed by Bangladesh Water Development Board (BWDB) for the protection of this area along the right bank from the devastating flood of the Padma river, but due to the continuous bank erosion, the flood embankment is also under danger and some parts of the embankment have already been washed out. This causes unmitigable losses of properties and unbearable sufferings to the people.

To save this area from the continuous bank erosion, RRI conducted physical model study to check the efficacy of proposed design of solid spur. **Figure 1** shows the plan of the solid spur.

The Padma river

The combined flows of Jamuna and Ganges rivers constitute the flow of the present Padma river. Before the avulsion of Jamuna river, the flow was a continuation of Ganges river only. The annual mean discharge is 28,000 m^3/s, and the bankfull discharge is about 75,000 m^3/s. The average size of the bed material is about 0.10mm. Geomorphologically, the river is still young. A flow regime analysis by FAP4 (1993)

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shows that it is now in a dynamic equilibrium. The planform of the river is a combination of the meandering and braiding type, indicating a wandering river. The variation of the total width of the river is quite high, ranging from 3.5km to 15km. The slope of the rivers is varying within a range of 8.5cm to 5cm per km. The bed material sizes are also varying from 0.20mm to 0.10mm. The dominant discharge is about 80,000m^3/s, which is slightly lower than that of the bankfull discharge of 82,500m^3/s (FAP24, 1996).

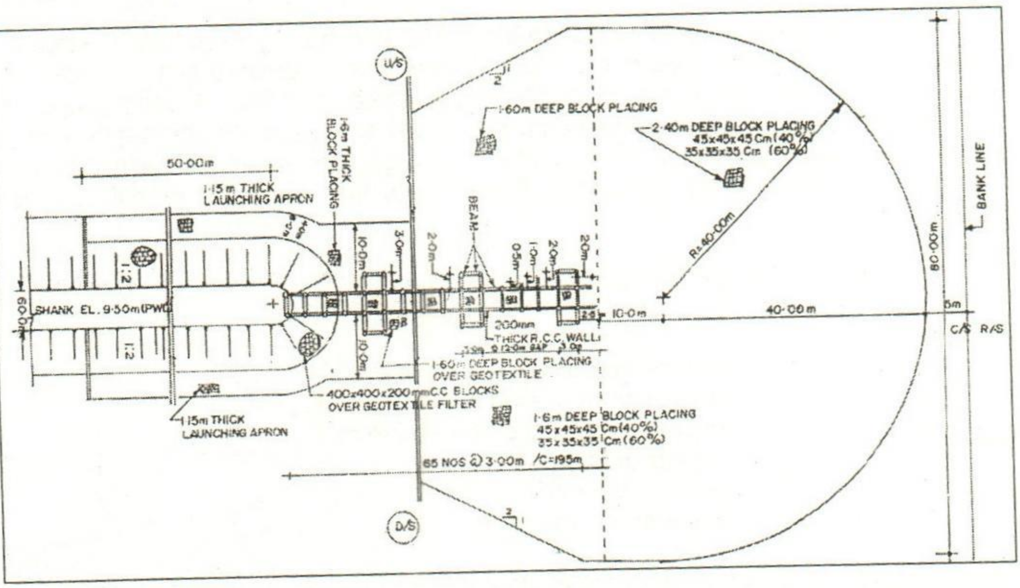

Figure 1 Plan of the RCC solid spur proposed by BWDB for the protection of charbhadrasion area from the erosion of the Padma river

Model design and construction

The model was designed according to the Froude's model law confirming the fulfillment of the purposes of the study using available modelling facilities at RRI. The model study was mainly aimed at simulating the flow pattern, flow velocity and scouring around the solid spurs to be constructed to protect the aforesaid area. To meet the requirement of the study, an undistorted movable bed sectional model of scale 1:100 was constructed to accommodate the study area. However, with this selected model scale the following simulation criteria were verified and found satisfactory:

- Sufficient water depth for precise measurement of velocity
- Rough turbulent flow in the model
- Sediment movement in the model

Critical velocity for sand movement in the model and prototype was calculated using the formula (Van Rijn, 1984)

$$V_{cr}=0.19(d_{50})^{0.1}\log(12h/3d_{90}) \qquad (1)$$

Ahmed (1953) stated that if $V_{cr}<V<2.5V_{cr}$ then local scour is a function of sand size and flow velocity. Scour depth becomes independent of flow velocity if $V/V_{cr}>2$ (FAP21/22, 1993). Here V and V_{cr} are average velocity and critical velocity respectively. It was found that the average velocity in the model was higher than the critical velocity for sediment movement in the model. Moreover, the model discharge was increased accordingly to achieve equilibrium scour earlier.

The model was constructed in an open-air model bed having fixed bed upstream & downstream and a mobile bed in the middle. There was a standard sharp crested weir at the upstream end of the model for the measurement of inflow. A point gauge was installed at a sufficient distance upstream of the weir to measure water depth over the weir for discharge calculation. The model discharge was measured with the sharp crested weir installed at the measuring flume, using the Rehbock's formula as given below:

$$Q=(0.403 + 0.053h_w/p)(2g)^{0.5}b\,[(h_w + u_w^2/2g)^{1.5} - (u_w^2/2g)^{1.5}] \quad (2)$$

Where,

g	=	acceleration due to gravity in m/s^2
h_w	=	head over the weir in m
P	=	height of the crest of the weir in m
u_w	=	velocity of approach in m/s
Q	=	discharge in m^3/s
b	=	length of the weir in m

A stilling pond was constructed at the upstream end of the model through which energy was dissipated. Hollow bricks were placed on the fixed bed across the upstream cross-section so that uniform flow was ensured in the model. Point gauges were installed to measure water level during model run. At the end of the model bed tailgates were installed to control the desired water level in the model.

Model calibration

Calibration was conducted to simulate the model with prototype condition. In a sectional model where whole width of the river is not considered, three boundary conditions are needed to be satisfied.

The boundary conditions for sectional model are:

- Upstream boundary condition at the inflow section of the model
- Downstream boundary condition at the out flow section of the model
- Lateral boundary condition is the location and alignment of the imaginary boundary wall constructed in the model as the left bank

The upstream boundary condition is the discharge distribution along the upstream limit of the model.

The comparison between computed and measured depth averaged flow velocities at C/S 4 is shown in **Figure 2**. From this figure, it is observed that the computed velocity was close to the measured velocity at the calibration section (C/S 4) during calibration of the model. So the model was calibrated with respect to prototype velocity.

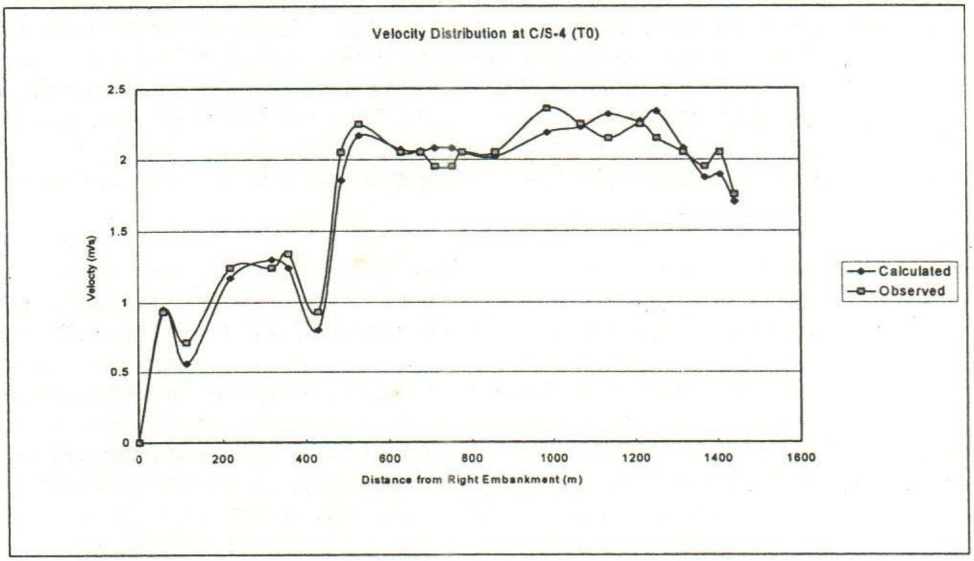

Figure 2 The comparison between computed and measured depth averaged flow velocities at C/S 4 during calibration of the model

The downstream boundary condition is the water level, which was maintained at the downstream of the model. So the water level found in the model was in good agreement with the prototype water level.

The lateral boundary condition is the location and alignment of the imaginary left boundary of the model. It is not the left bank of the river, but it is a flow line of the prototype in order to represent a portion of the river where discharge is constant. The following criteria were considered to fix up the lateral boundary:

The width of the model should be sufficient to prevent any influence by the solid spurs on this flow line.
The curvature of the boundary wall should be as smooth as possible to avoid flow separation (FAP-21/22)

No influence of the solid spurs on the left boundary as well as no flow separation at the left bank was observed during calibration of the model.

Test description

Considering RCC solid spur one calibration test (T0) plus three different application tests (T1, T2 & T3) were conducted in the model study. Calibration test contributed to the calibration of the model with Froude discharge. Each application test was run with Froude discharge to determine flow velocity, flow lines etc and that of scour discharge to determine the scour depth around the solid spur.

Calibration test was carried out with existing condition i.e. without any proposed spur & with Froude discharge. During calibration test the near bank depth averaged flow velocities were measured in the model at different cross-sections and flow field by float tracking was recorded. The near bank depth averaged flow velocities and flow lines in the calibration test have been used as a reference value to be compared with each application test. The near bank velocities measured during calibration test are shown in Table 1. From this table it is observed that the magnitude of near bank velocities at some sections was higher than the erodible velocity. From recorded float tracking in this test it shows that the flow lines have a tendency of attacking the bank. Near bank velocities and flow lines indicate that river training structure is essential to combat river bank erosion.

Considering different location of solid spurs three application tests were carried out. Then the near bank velocity and the diverting amount of flow lines from the bankline by the solid spurs in a series in each test have been compared with that of the calibration test. Among the different application tests, Test T2 provides better performance because of the flow lines have been diverted significantly away from the bankline and near bank velocities were fairly reduced due to the suitable location of the series of solid spur adopted in this test. So the solid spurs in Test T2 have been recommended to be constructed on the flood plain. No significant scour was occurred around the spurs. The maximum velocity around the head of the spurs was found about 2 m/s. The specification of spurs adopted in the recommended test is shown in Table 2.

Table 1 Near bank depth averaged flow velocities during calibration Test (T0)

C/S No.	Near bank velocity (m/s)																			
	Distance from right embankment																			
	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000
4	0.6	0.7	0.8	0.8	1.4	1.3	1.0	0.6	0.9	2.1	1.6	1.2	1.8	1.6	1.6	2.1	2.1	2.3	2.4	2.3
5	1.0	1.0	1.1	1.6	1.4	1.4	1.3	2.5	2.1	1.9	1.7	2.2	2.1	2.4	2.3	1.9	2.5	2.3	2.3	2.4
6	1.2	1.6	1.3	1.4	1.7	1.4	1.9	2.5	2.3	2.1	1.8	2.3	1.9	2.3	2.1	2.4	2.5	2.4	2.1	2.2
7	0.0	1.1	1.4	2.2	2.7	2.5	2.6	2.1	2.3	2.4	2.4	2.5	2.5	2.6	2.6	2.8	2.7	2.6	2.3	2.5
8	0.0	0.0	0.5	0.7	1.4	2.4	2.5	1.8	1.9	1.9	2.1	2.1	1.8	1.7	2.3	2.4	2.5	2.5	2.3	1.9
9	0.0	0.0	0.6	1.4	2.3	2.8	2.6	2.7	2.5	2.3	2.4	2.4	2.2	2.5	2.9	2.5	2.8	2.7	2.4	2.1
10	0.0	0.0	0.7	1.4	2.5	2.9	2.2	1.9	2.1	2.2	1.9	2.5	2.3	2.4	2.4	2.5	2.4	2.3	2.4	2.1
11	0.0	0.0	0.6	0.8	2.5	2.3	2.4	2.4	2.5	2.1	2.4	2.4	2.5	2.4	2.3	2.5	2.3	2.8	2.6	2.1
12	0.0	0.0	0.5	0.8	2.1	2.9	2.2	1.9	1.4	1.8	2.1	1.7	1.6	1.7	1.8	1.9	2.3	2.1	1.9	2.1
13	0.0	0.0	0.4	0.7	1.0	1.2	2.9	2.3	1.6	1.2	1.8	1.5	1.4	1.5	1.9	1.8	1.8	1.6	2.2	2.1
14	0.0	0.0	0.4	0.8	1.6	2.5	2.4	2.3	1.8	2.1	2.2	2.3	2.1	2.4	2.3	1.9	2.3	2.8	2.3	2.4
15	0.0	0.0	0.5	0.7	1.7	1.9	1.6	2.3	1.7	1.8	1.2	1.6	1.2	1.6	1.8	2.2	2.1	2.3	1.6	1.5
16	0.0	0.0	0.0	0.8	1.2	2.4	1.9	1.6	1.7	1.9	1.6	1.4	1.3	1.2	1.3	1.3	1.2	1.3	1.3	1.4
17	0.0	0.5	1.0	1.0	2.0	2.6	1.8	1.9	1.7	1.6	1.8	1.8	1.7	1.9	1.9	1.8	1.9	1.8	1.8	2.3
18	0.0	0.0	0.5	1.0	1.0	2.3	2.5	2.5	2.3	2.3	2.3	2.3	2.0	2.1	2.0	2.0	2.1	2.0	2.5	2.3
19	0.0	0.0	0.0	0.0	0.5	1.0	1.3	2.3	1.8	1.8	2.0	1.8	2.1	1.9	1.8	1.5	1.5	1.6	1.9	1.8
20	0.0	0.0	0.0	0.0	0.5	1.0	1.3	2.4	2.5	2.7	2.6	2.1	2.2	2.2	2.1	1.9	2.1	2.1	2.1	2.0
21	0.0	0.0	0.0	0.0	0.6	1.2	1.7	2.2	2.3	2.9	2.7	2.1	2.2	2.3	2.2	2.1	2.0	1.9	2.1	2.0
22	0.0	0.0	0.7	1.2	1.5	2.8	2.8	2.6	2.7	2.5	2.6	2.3	1.9	1.9	2.0	2.0	2.3	2.3	2.2	2.2
23	0.0	0.5	0.7	2.3	2.9	3.5	2.9	2.9	3.1	2.8	2.7	2.2	1.8	1.9	2.0	2.3	2.3	2.0	1.8	1.9

Table 1 (cont.)

C/S No.	Near bank velocity (m/s)																			
	Distance from right embankment																			
	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000
24	0.0	0.0	0.8	0.8	1.9	2.9	2.7	2.7	2.8	3.0	2.9	2.6	2.6	2.3	2.3	2.2	2.5	1.9	1.5	1.9
25	0.0	0.0	0.0	0.0	0.6	2.1	2.9	2.8	2.7	2.8	2.4	2.5	2.7	2.2	2.1	2.2	2.0	2.1	1.5	1.5
26	0.0	0.0	0.9	1.5	2.3	1.8	3.1	3.3	3.3	2.9	3.0	2.9	2.6	2.5	2.6	2.9	2.0	1.8	2.0	2.1
27	0.0	0.0	0.0	0.8	1.8	2.7	2.7	2.3	2.9	3.3	2.5	2.3	2.8	2.5	1.4	1.3	1.1	1.4	1.3	1.4
28	0.0	0.0	0.5	1.2	1.5	3.2	3.3	3.3	3.4	3.3	3.4	3.7	3.0	2.5	2.4	2.8	2.8	2.3	2.5	2.2
29	0.0	0.0	0.7	1.5	2.2	3.4	3.2	2.3	3.1	3.0	2.3	2.8	2.9	2.4	2.6	2.1	2.4	2.4	2.5	2.4
30	0.0	0.0	0.4	0.8	1.5	2.8	2.9	3.0	3.2	3.5	3.4	2.9	2.8	2.9	2.9	2.9	2.8	2.7	2.9	3.3
31	0.0	0.0	0.0	0.0	0.0	0.0	1.8	2.3	2.5	2.6	2.2	2.4	2.6	2.7	2.4	2.8	2.6	1.9	2.4	2.5
32	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	2.5	2.6	1.9	2.6	2.6	2.8	2.9	2.8	2.4	2.5
33	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	2.5	3.0	2.5	2.7	2.7	2.8	3.0
34	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.8	2.8	2.6	2.4	2.8	2.3
35	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	1.9	2.4	2.8	2.6	2.7
36	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	2.1	2.8	2.7	2.8	2.3	2.5
37	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.6	1.9	2.5	2.7	2.8	2.7	2.9
38	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	2.3	2.4	2.7	2.8	2.6	3.3
39	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.7	3.1	3.0	2.9	2.9	2.6	2.6
40	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	2.3	2.5	2.7	2.6	2.5	2.3	2.3
41	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	2.3	2.7	2.6	2.5	2.5	2.3	2.4	2.3
42	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	2.3	2.3	2.5	2.7	2.6	2.5	2.3
43	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.7	2.4	2.4	2.3	2.3	2.1	2.2
44	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.9	1.8	2.1	1.9	2.2	2.2	2.1	2.1
45	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.7	1.8	2.1	2.3	2.3	2.3	2.2	1.8
46	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0	2.7	2.2	2.2	1.8	2.3	2.4	1.9	1.4
47	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.3	1.6	1.5	1.7	1.6	1.6	1.5	1.5
48	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	2.4	2.5	2.4	2.2	1.7	1.8	2.3
49	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.4	1.5	1.7	1.9	2.3	1.5	1.9	1.7	2.1
50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.6	2.1	1.9	1.7	1.9	1.9	1.6
51	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.3	1.8	2.4	1.9	1.8	1.7	1.7
52	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	2.1	2.2	2.4	2.2	1.9	1.7	1.8	1.9

From **Table 1** it is observed that the magnitude of near bank velocities was higher than the erodible velocity. Near bank velocities indicate that river training structure is essential to combat bank erosion.

Table 2 Specification of solid spurs adopted in the recommended test (Test T2)

Spur No.	Alignment & location of spur	Length of spur (m)	Length of shank (m)	Total length (m)
S1	Projected from right embankment along C/S 5 (Ch 0+800)	197.5	76.5	274
S2	Projected from right embankment along C/S 10 (Ch 1+800)	152.0	-	152
S3	Projected from right embankment along C/S 14 (Ch 2+600)	185.0	-	185
S4	Projected from right embankment along C/S 18 (Ch 3+400)	197.5	4.5	202
S5	Projected from right embankment along C/S 22 (Ch 4+200)	172.0	-	172
S6	Projected from right embankment along C/S 26 (Ch 5+000)	197.5	32.5	230
S7	Projected from right embankment along C/S 31 (Ch 6+000)	197.5	153	350
S8	Projected from right embankment along C/S 36 (Ch 7+000)	197.5	472	669
S9	Projected from right embankment along C/S 41 (Ch 8+000)	197.5	396	594
S10	Projected from right embankment along C/S 47 (Ch 9+200)	197.5	423	621
S11	Projected from right embankment along C/S 51 (Ch 10+000)	197.5	506	703

Recommendations

Considering the proposed series of solid spur an extensive model study has been done to investigate the effectiveness of the series of solid spur and to provide data for the finalization of the design. The model study was conducted on the basis of river bathymetry of February 2001. After this period bed level, bank line, thalweg and morphology of river might be changed and due care should be given to asses these changes before construction of solid spurs in the field. The following recommendations are made based on the test results of model study considering solid spur:

- During model study with solid spur it is observed that total eleven solid spurs are necessary to protect about 10km area from C/S 3 to C/S 52. At least three consecutive solid spurs from the upstream to be constructed in one year for joint hydrodynamic effect.

- From model findings it appears that the first solid spur has fallen under severe attack, so during construction of first solid spur special care should be taken to strengthen the solid spur.
- The specification of solid spurs in the recommended test is given in Table 2. The recommended location and orientation of solid spurs are shown in Figure 3.
- Different sizes of block should be kept ready stock for emergency dumping, if necessary.

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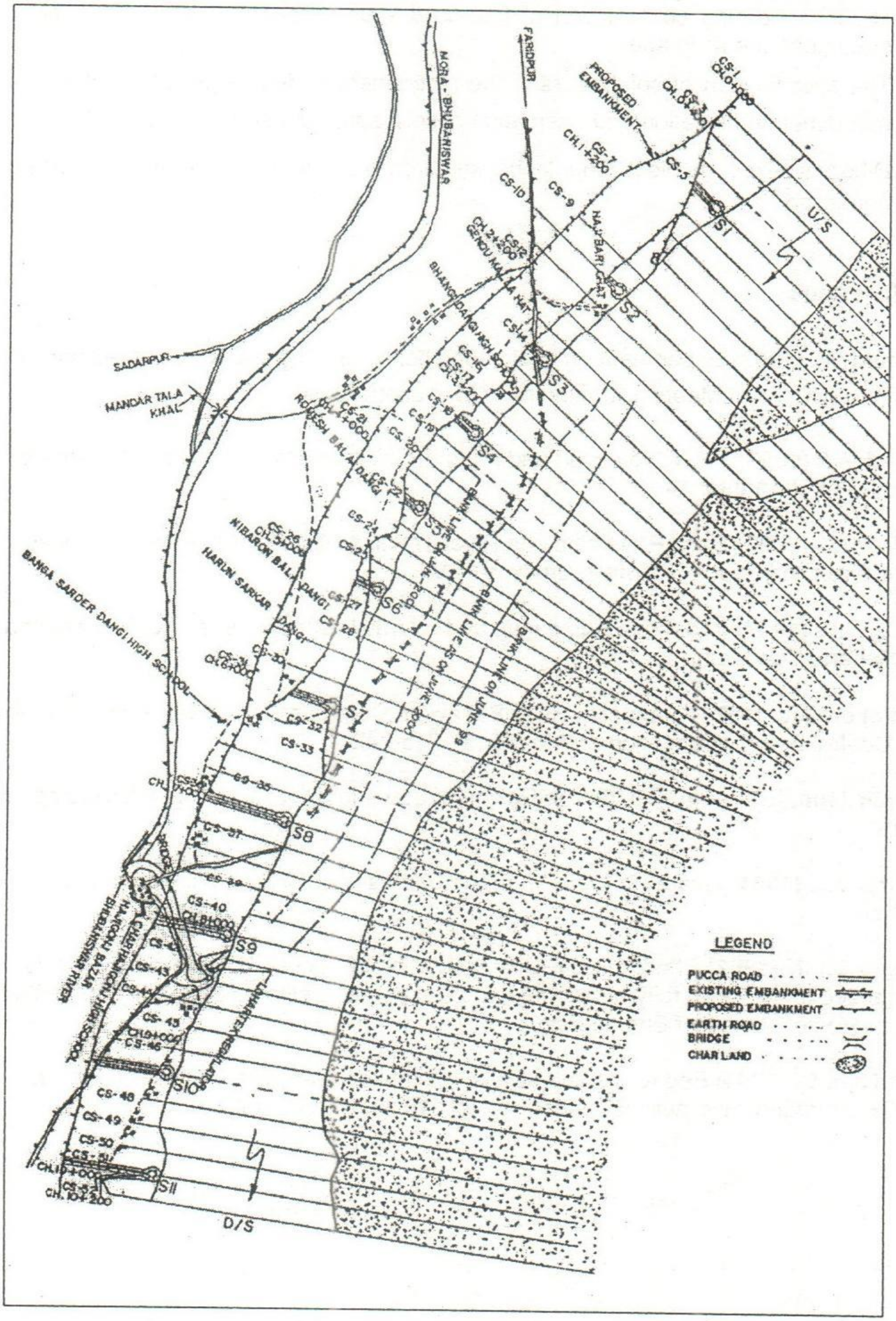

Figure 3 The recommended orientation and location of solid spurs in the study area

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COMPARATIVE STUDY OF SOIL SAMPLES FOR QUICK CONSOLIDATION AND NORMAL CONSOLIDATION TESTS

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Abstract

Investigation of soil characteristics are very important for construction of various types of structures. There are various tests for determination of different parameters of soil according to soil types. The settlement characteristic is one of the most important parameters of cohesive soil and it is mainly determined by consolidation test. The main purpose of consolidation test on the soil samples is to obtain necessary information on the compressible properties of a saturated soil for use in determining the magnitude and rate of settlement of the structures. Hence a good knowledge of the field condition as well as compressibility and permeability characteristics of soil is very essential for better understanding of the consolidation behavior of soil deposit. Consolidation test of soil is very important for construction works but the testing technique is time consuming. It needs about 8 days to carryout the test. So for reduction of testing time, a research study was undertaken to establish quick consolidation method. In this research study, normal consolidation and quick consolidation tests were performed consecutively on the large number of undisturbed soil samples of different ingredients. Co-factor for compression index (Cc) for such soil types were found out comparing results obtained in quick consolidation and normal consolidation tests. The finding of this study is that the quick consolidation method is approximately valid in case of inorganic CLAY and SILT mixed with trace fine sand. The main advantage of using this method is that it will reduce the normal testing time

Introduction

Bangladesh is a developing country and many construction works are going on. As a reverine country the extensive flood plain of these rivers and their numerous tributaries and distributaries are the main physiographic phenomena of these alluvial rivers (H.Mazumder et al. 2000).

As a result of extensive flood many water control structures like embankments, regulators, dykes, culverts etc. are destroyed. Hundreds of thousands of people become homeless and multicrore national projects become threatened almost every year due to bank erosion (F.Karim et al. 1998). So. for overcome this situation it is needed to take up the national projects for reconstruction of the water control structure and construction of new structure .

On the other hand, Bangladesh is a densely populated country. There are no enough space for accommodation in urban areas. So, for accommodation of large number of

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population in small space, construction projects of multistoried building have been undertaken in urban areas.

For the construction works in different types of projects as mentioned above, various engineering parameters of soil are determined in which consolidation parameter is an important factor. It has significant effects on the vertical stress exerted by the column and on the reduction of settlement of the foundation soil (Juran and A. Guermaz, 1988).

To know the consolidation behavior of soil deposits,knowledge of the field condition as well as the compressibility and permeability characteristics of the material is essential. These characteristics are usually presented in the form of void ratio-effective stress and void ratio- permeability relationships, which are obtained experimentally (Jin-Chun Liu et. al 1991)

The consolidation of a soil deposit takes place in three stages :

- Initial consolidation
- Primary consolidation
- Secondary consolidation

In case of most inorganic soils, secondary consolidation is generally small (K.R. Arora, 1992). Secondary consolidation effects are often disregarded for inorganic clays, in which primary consolidation phase is responsible for the majority of settlement (K.H. Head, 1982).

If the primary consolidation stage is clearly completed by the time 100 min, it would be feasible to apply a second load increment almost immediately instead waiting for 24 hours. In that case it is possible to complete several load increments within one day (K.H. Head ,1982). From this point of view, in this research program the time for settlement (deformation) for each load in quick consolidation method had been considered 100 minutes.

The consolidation test of soil is essential for construction works but the testing technique is time consuming. So for the purpose of reduction of testing time and finding out co-factor for compression index the concerned research study was undertaken to establish quick consolidation method.

Literature review

Geological background

The landscape of Bangladesh is mainly monotonous flat plain. From geological point of view, most of Bangladesh is an extremely flat delta which consists of a large alluvial basin floored primarily with Quaternary sediments deposited by the Ganges and Brahmaputra rivers and their numerous associated streams and distributaries. The three major physiographic units namely Hill formations of Sylhet in the north-east and of Chittagong Hill tracts in the south-east, uplifted Pleistocene Terraces and recent flood-plain and piedmont alluvium, which occupies roughly seventy percent of the total land area of Bangladesh (Hunt, 1976). The recent floodplain deposits are again differentiated into meander flood-plain deposits, depending on the environmental conditions that existed at the time of deposition.

A.M. Zahural Islam and P.K. Roy(1991) discussed that the soils of Bangladesh comprised of alluvial deposit varying from gravel to fine grained soil material and the stratified arrangement of soil materials mainly depends on the fluctuation of the stream velocity of the main rivers. Thus in the upper region of Bangladesh the soil is a mixture of coarse and fine grained soils like boulders, gravels, sand, silt and clay and in the lower region obviously a deposit of fine grained soils like sand, silt and clay. In the western region of the country the soils mainly consist of sand, silt and clay deposits.

Prof. Safiullah (1994) described that the classification of ingredients of constituents of soil is of utmost importance. Because different types of soil contain different ingredients in varying amount for which its properties vary.

On the basis of geological formation of the soils of Bangladesh their general characteristics are given below:

Table-1: Characteristics of Bangladeshi Soil (Safiullah, 1994)

Soil Type	General characteristics
Hill Soils	Variable soil types which are function of underlying geology. Frequently sandy clays and clays grade into disintegrated rock at shallow depths.
Raised alluvial terrace deposits	Comprises relatively homogeneous clay known as Modhupur clay and Barind clay. (LL=30-40%; Ip= 12-50%). Variable depth underlain by fine to medium uniformly graded sand.
Himalayan piedmont deposits	Mainly sandy silt in higher areas and silty clays in basin areas but often underlying fine sands at shallow depth
Alluvial flood plain deposits	Locally variable but in general silts and silty clays. Silt size predominant. (LL=20-50%, Ip= 4-30%). Fine sands abound at depths and close to rivers. Contain mica.
Depression deposits	In the south alternating organic clay deposits overlying clay at depth. Elsewhere, predominantly silty clays and clays, (LL=30-40%, Ip=10-16%).
Estuarine and flood plain deposits	Generally silt and silty clays. Acid sulfate soils found near coast. Organic soils close to surface in some places. Widely varies in consistency and water content.

Where, LL= Liquid limit, Ip= Plasticity Index.

Physical properties of Bangladeshi soils

M, Serajuddin(1998) explained that colors of the soils usually range from light grey to dark grey and (often) black, from light brown to brown or reddish and occasionally a combination of them. Light grey to dark grey are more frequently obtained than those of other colors. Soils with bluish and greenish colors are also encountered sometimes .

Some engineering properties of soils in relation to consolidation parameter

K.R. Arora(1992) mentioned that the compression index is related to its index properties, especially the liquid limit. Terzaghi and Peck gave the following empirical relationship for clays of low to medium sensitivity ($S_t < 4$)

C_c= 0.009 (w_L –10) ----- For undisturbed soils

C_c=0.007 (w_L -10) ------ For remolded soils

Where, w_L= Liquid limit (%)

The compression index is also related to the in situ void ratio e_0 or water content w_o,

$$C_c= 0.54(e_0-0.35)$$

$$C_c= 0.0054(2.6w_0-35)$$

M, Serajuddin (1998) stated that the compression index (C_c) of compressible clays and silts has some empirical relations with liquid limit (w_L), initial void ratio (e_0) and natural water content (w_N). Serajuddin and Ahmed (1967) co-related C_c with w_L and e_0 of a large number of undisterbed plastic silt and clay soil samples of different areas of Bangladesh and obtained the following empirical equations:

$$C_c= 0.0078(w_L-14\%)$$

$$C_c= 0.44(e_0-0.30)$$

A general survey by Serajuddin (1964 and 1969) on the engineering aspects of soils of Bangladesh suggested the following two relationships for Cc with e_0 and w_N for fine grained soil of the coastal embankment areas:

$$C_c=0.50(e_0-0.50)$$

$$C_c= 0.0135(w_N-20\%)$$

Another correlation by Serajuddin and Ahmed (1982) with additional test data from cohesive fine grained soils occurring within about 7m from the ground surface of the different areas of the country suggested the relationship:

$$C_c= 0.047(e_0-0.46)$$

K.H. Head (1982) obtained the usual range of values of the co-efficient of consolidation (C_v) together with the values of compression index (C_c) from laboratory consolidation tests as shown in Table-1:

Table-1 : Typical range of values of co-efficient of consolidation and compression index for inorganic soils (K.H.Head1982)

Soil type	Plasticity index (range)	Coefficient of consolidation $C_v (m^2 / year)$		Compression index C_c
		undisturbed	remolded	
Clays-montmorillonite high plasticity	greater than 25	0.1-1	About 25-50% of undisturbed values	upto 2.6
medium plasticity	25-15	1-10		
low plasticity	15 or less	10-100		
Silts		above 100		

Methodology

For determination of C_c value, quick consolidation test and normal consolidation test were performed for same undisturbed soil samples (visually same) of different location of Bangladesh. The undisturbed soil samples were received from Bangladesh Water Development Board (BWDB) collected in connection with different project of BWDB.

The sample was trimmed to fit a cylindrical container and loaded up to 24 hours interval for each load increment for laboratory normal consolidation test and loaded up to 100 minutes in case of quick consolidation test for each load increment. For a given load increment, deformation reading were taken from the respective dial gauge for different time intervals.

Consolidation parameters like void ratio, natural moisture content, liquid limit, plastic limit, specific gravity were also determined for the same undisturbed samples

Presentation of results

Graphical presentation of dial reading (Deformation) vs. log time for typical clay soil for normal and quick consolidation tests for each load increments has been shown in **Figure 1**.. Deformations (D_0, D_{100} & D_{50}) and the corresponding t_{50} as shown in Fig-1 has been calculated from the graph. In case of silt samples, as it is difficult to evaluate the D_0 point from the log-time plot, so the settlement is plotted against square root of time in minutes which has been shown in **Fig-2**.

Void ratio versus log pressure and co-efficient of consolidation (C_v) versus log pressure for the loading and unloading stages for both quick and normal consolidation tests have been shown **in Fig-3** and **Fig-4** for clay and silt samples respectively.

The compression index C_C is determined from the tangent of the slope at the void ratio versus log- pressure curve by using following equation,

$$C_c = - e / \log p/p_0 = e / \log \overline{\sigma}/\overline{\sigma_0}$$

Where,

$\overline{\sigma_0}$= initial effective stress
$\overline{\sigma}$= final effective stress

Δe= change in void ratio

Co-factor for compression index for different soil types have been found out comparing the C_c values obtained in normal and quick consolidation tests and have presented graphically in Fig-6. The parameter related to consolidation tests and concerned parameter ranges according to soil types have been shown in Table-2 and Table-3 respectively. Compression index versus soil types for both normal and quick consolidation test has been shown in bar graph in Fig-5.

Discussion of results

It is observed from Table –2 that the ingredients of undisturbed soil samples tested by two methods viz. normal and quick consolidation are not homogeneous. Table-3 shows the classification of soil types according to ingredients of tested samples and their ranges of co-factors. From Table-3 it is observed that co-factors for Cc values for all soil types are almost unity except the value for very soft to soft organic soils.

From graphical presentation of time settlement curve, it has been found that primary consolidation of soil in case of quick consolidation reaches as a same sequence as normal consolidation test, which has been shown in Fig. 1 & 2. Only for time reduction of load increment in case of quick consolidation test the deformation reading has varied from normal consolidation test as a result void ratio and compression index has also varied.

It is observed from Fig. 6 that the co-factor of compression index Cc has slightly varied and it is due to soil types, stiffness and for ingredients of soil. Natural moisture content and plasticity of soil are also responsible for the variation of compression index and it is due to field condition i.e. ground water level.

Table-2 : Soil type and test results of natural moisture content, liquid limit, plasticity index, specific gravity and compression index

Soil type	Av. N.M.C in %	LL in %	PI in %	Sp. Gr.	Normal consolidation test		Quick consolidation test	
					Initial.N.M.C in %	C_c	Initial N.M.C in %	C_c
Soft CLAY trace org. trace fine sand H.P	54.54	60	30	2.668	54.53	0.496	54.46	0.440
Ver soft CLAY trace org. trace fine sand H.P	59.61	61	31	2.667	56.91	0.423	58.19	0.447
Soft CLAY trace org. trace fine sand H.P	43.24	56	29	2.672	43.70	0.337	43.63	0.314
Soft CLAY trace fine sand H.P	43.47	58	29	2.678	43.06	0.395	42.70	0.375
Very soft CLAY trace organic trace fine sand H.P	53.12	60	30	2.669	54.08	0.515	54.47	0.495
-Do_	52.56	55	28	2.671	48.8	.352	46.8	0.312
Soft CLAY trace fine sand H.P	50.82	56	29	2.676	47.60	0.354	44.50	0.313
-Do-	46.71	63	33	2.682	47.44	0.356	48.34	0.372

Table-2 (Continued)

Soil type	Av. N.M.C in %	LL in %	PI in %	Sp. Gr.	Normal consolidation test		Quick consolidation test	
					Initial.N.M.C in %	C_c	Initial N.M.C in %	C_c
Very soft CLAY trace fine sand M.P	40.54	45	20	2.678	44.98	0.346	40.66	0.422
Very soft CLAY little organic little fine sand H.P	46.11	59	31	2.5616	52.8	0.544	44.39	0.348
Very soft CLAY little organic trace fine sand H.P	56.11	64	30	2.569	71.91	0.484	54.36	0.686
Very soft ORGANIC CLAY trace fine sand M. to H.P	46.36	86	45	2.556	61.13	0.619	62.60	0.590
Soft CLAY trace organic trace fine sand H.P	46.72	55	27	2.644	40.99	0.330	37.26	0.280
Soft CLAY trace fine sand H.P	61.02	64	32	2.609	55.5	0.520	56.65	0.547
Very soft ORGANIC CLAY trace fine sand M. to H.P	80.48	85	42	2.531	66.91	0.563	79.31	0.697
Very soft ORGANIC CLAY trace fine sand M. to H.P	65.23	77	40	2.557	51.07	0.375	57.53	0.365
Very soft CLAY trace fine sand H.P	51.97	58	29	2.585	50.55	0.300	51.89	0.295
-Do-	51.53	59	29	2.578		0.508	51.58	0.401
Soft SILT trace fine sand M.C	43.42	42	16	2.674	38.12	0.260	38.86	0.25
-Do-	23.45	40	13	2.671	37.40	0.235	36.50	0.245
Very soft CLAY trace fine sand H.P	47.2	69	31	2.6332	45.52	0.394	47.53	0.337
Soft CLAY trace fine sand H.P	44.56	55	29	2.5616	46.09	0.390	47.64	0.402
Soft CLAY some fine sand M.P	34.42	43	21	2.6435	30.79	0.261	32.71	0.257
Medium stiff FINE SAND AND CLAY M.P	26.37	40	19	2.6378	37.61	0.385	36.85	0.384
Soft CLAY trace fine sand H.P	41.53	54	28	2.5665	25.76	0.160	25.91	0.130
Soft CLAY some fine sand M.P	44.81	45	22	2.5608	42	0.425	43.92	0.395

Notations used in the Table-2:

N.M.C = Natural moisture content

LL = Liquid limit

PI = Plasticity index

Sp.Gr. = Specific gravity

C_c = Compression index

H.P. = High plastic

M. to H. P = Medium to high plasticity

M.C. = Medium compress

M.P = Medium plasticity

Table-3 : Range of consolidation parameters and co-factor according to soil types

Soil type	Sp.Gr.	Av. M.C in %	LL in %	PI in %	Normal Test			Quick Test			Co-factor C_f
					N.M.C in %	initial void ratio	Cc	N.M.C in %	initial void ratio	Cc	
Very soft to medium stiff CLAY mixed with trace fine sand and plasticity varies from medium to high plastic	2.5608-2.6820	26-55	40-64	19-32	26-52	0.8935-1.4514	0.160-0.520	26-57	0.6316-1.6961	0.130-0.547	0.819-1.231
Very soft to soft CLAY mixed with trace to little organic and fine sand and plasticity is high plastic	2.561-2.672	39-63	55-64	27-31	41-72	0.6832-2.0057	0.330-0.544	37-54	1.0064-1.4925	0.280-0.686	0.820-1.563
Very soft to soft ORGANIC CLAY mixed with trace fine sand and plasticity is medium to high plastic	2.531-2.557	55-73	77-86	40-45	51-67	1.3294-1.7071	0.563-0.619	58-79	1.0600-2.1370	0.590-0.697	0.808-1.049
Soft SILT mixed wtih trace fine sand and compressibility is medium compress	2.671-2.674	37-39	40-42	13-16	37-38	1.0661-1.1.48	0.235-0.26	37-39	1.076	0.245-0.25	0.959-1.04

Conclusions and recommendations

Results obtained from the outcome of the study the following conclusion and recommendation are drawn.

- The ingredients of soil greatly influence the soil parameter as well as consolidation parameter.
- The quick consolidation method is approximately valid in case of very soft to medium stiff inorganic CLAY mixed with trace fine sand (Plasticity: Medium to high plastic).
- The method is also valid approximately in case of soft SILT mixed with trace fine sand (Compressibility : Medium compress).
- The results of Cc value in quick consolidation method differs greatly from normal consolidation method in case of very soft to soft organic CLAY mixed with trace fine sand.
- The advantage of quick consolidation are that it will reduce the testing time
- Naturally soil strata and ingredient is not homogeneous. So, to establish "quick consolidation method" further study on more and more samples is recommended.

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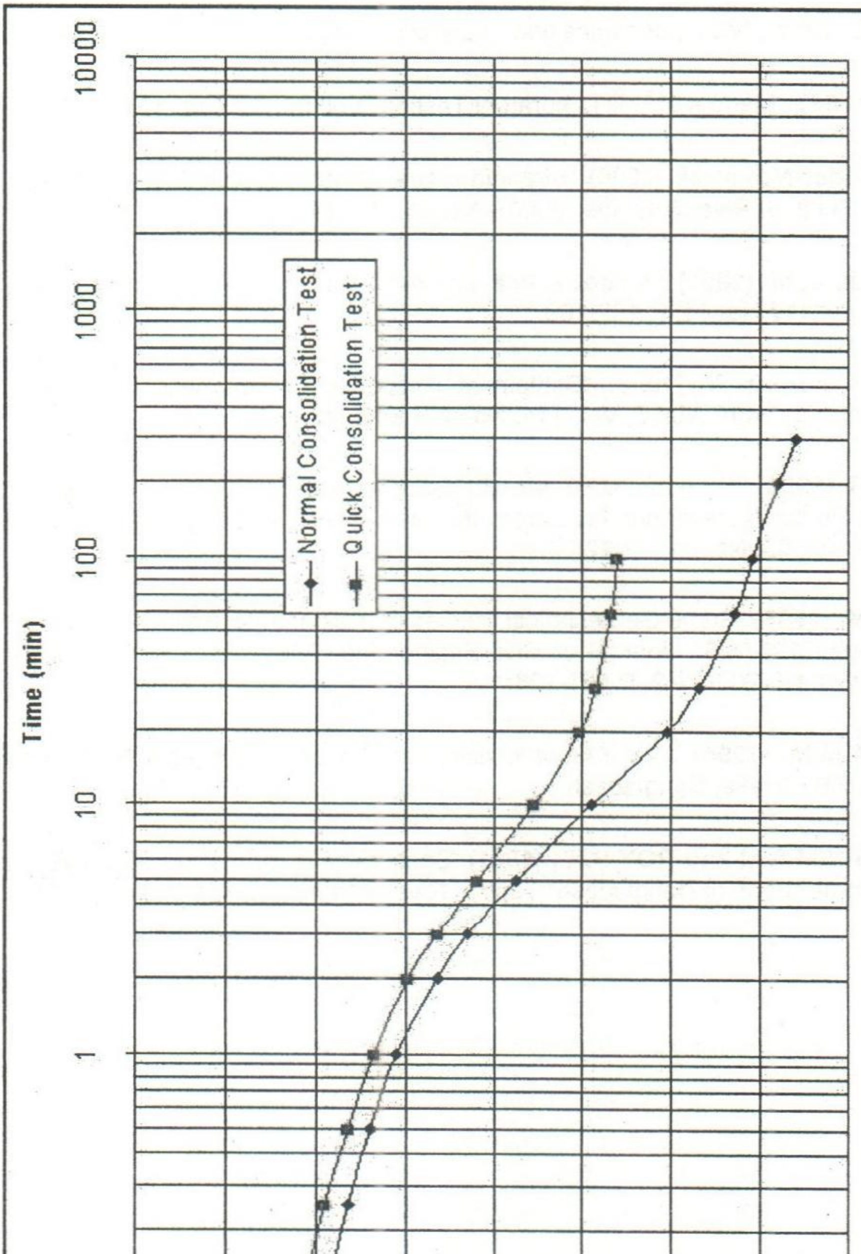

Figure-1 : Time- Deformation Curve (Clay Soil)_

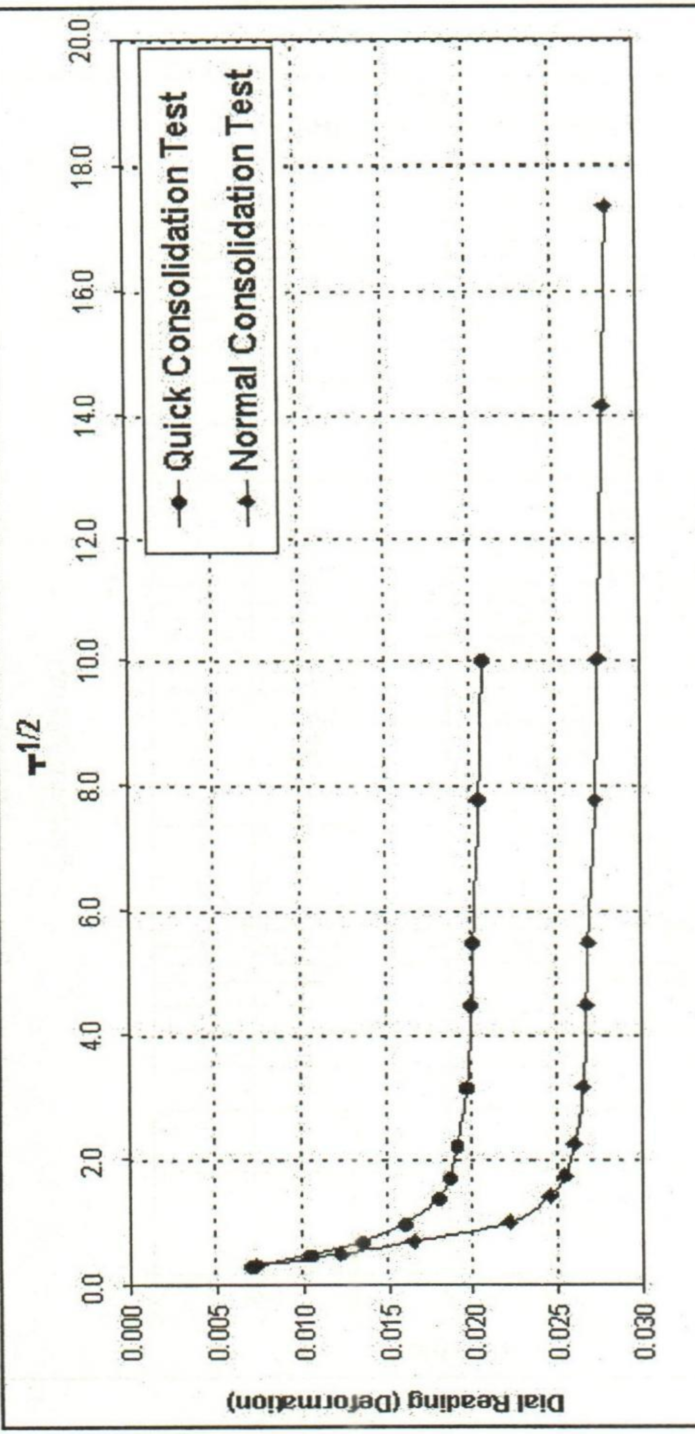

Fig.: 2 Square root time VS Deformation Curve (Silty Soil)

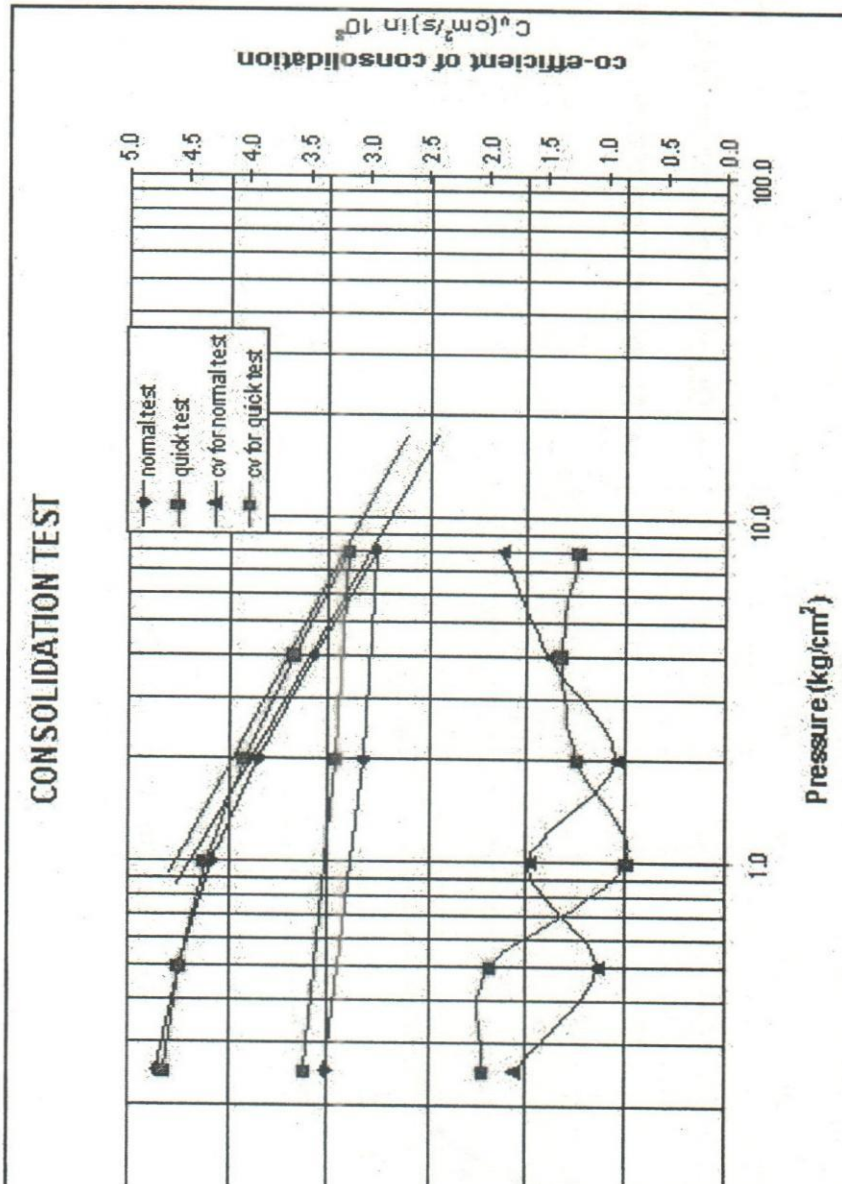

Fig. 3: Void Ratio Vs Pressure and C_v Vs Pressure Curve (Clay Soil)

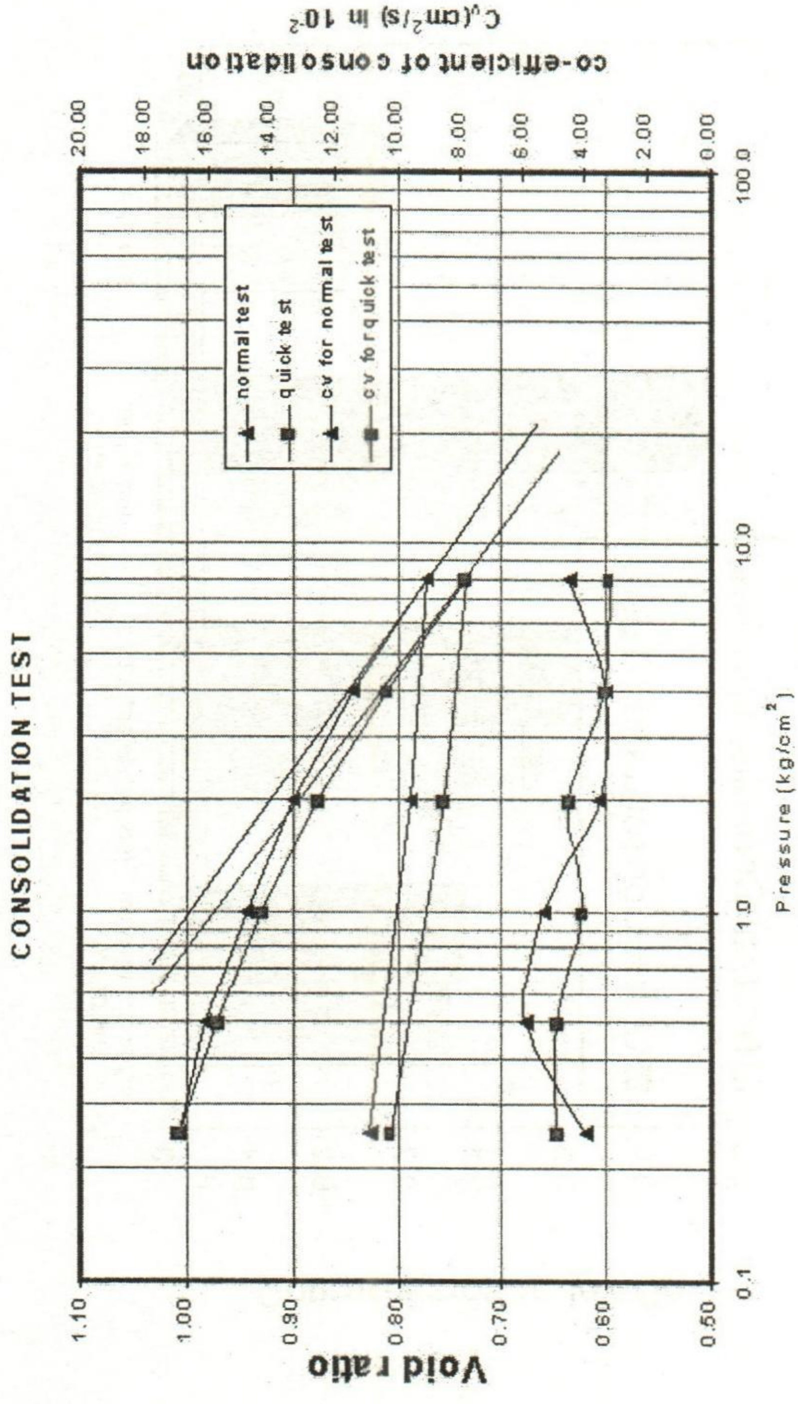

Fig. 4: Void Ratio Vs Pressure and Cv Vs Pressure Curve for typical Silty Soil

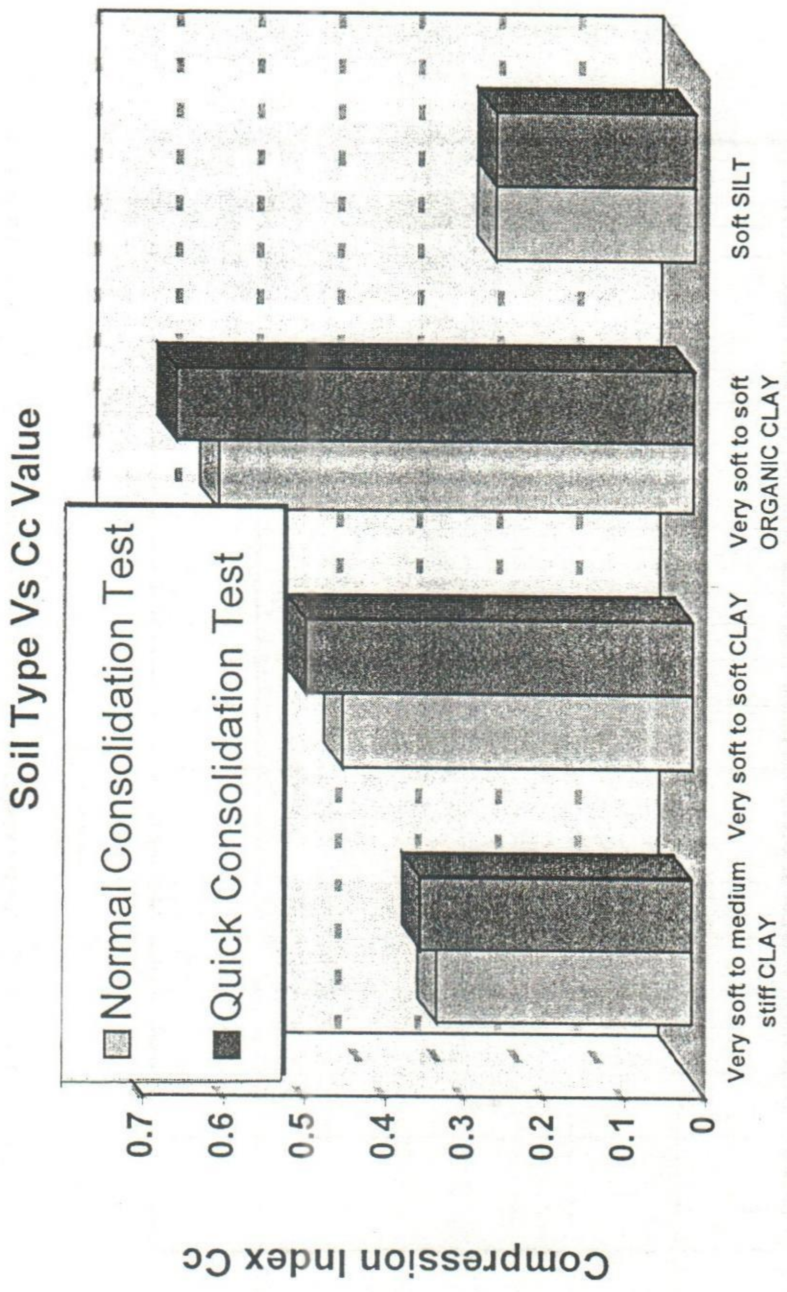

Fig. 5: Soil Type Vs Cc Value for Different Types of Soil

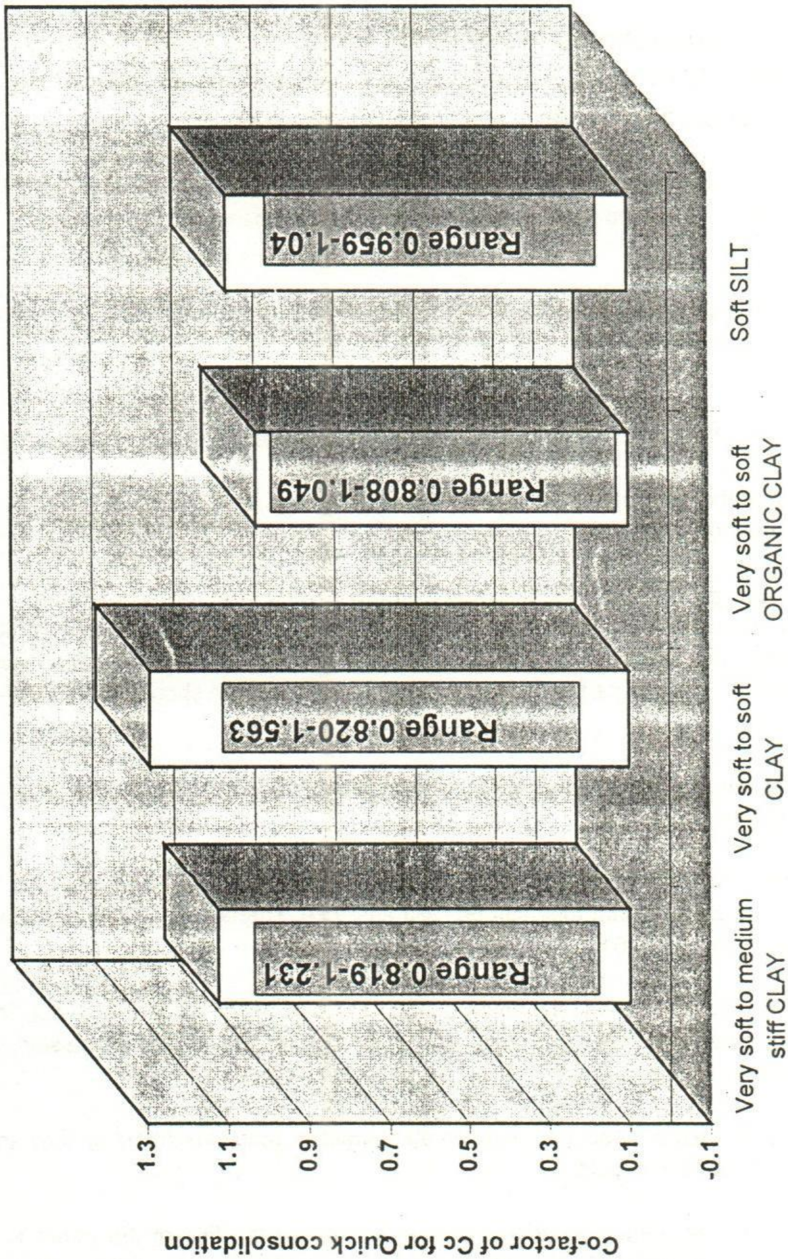

Fig. 6 : Range of Co-factor of Cc for quick consolodation for different types of soil

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