Vol. 05 No. 01 October 1998

TECHNICAL JOURNAL

RIVER RESEARCH INSTITUTE
FARIDPUR, BANGLADESH

TECHNICAL

JOURNAL

Vol. 05 No.01 October 1998

RIVER RESEARCH INSTITUTE
FARIDPUR, BANGLADESH

TECHNICAL JOURNAL
RIVER RESEARCH INSTITUTE, FARIDPUR
October 1998

TECHNICAL PAPERS

EVALUATION OF DISCHARGE CO-EFFICIENT OF BROAD-CRESTED MASONRY WEIRS

Md. Abul Ala Moududi[1], A M M Motaher Ahmed[1], Md. Manjurul Haque[1]
Md. Mutaher Hussain[1], Md. Monirul Islam[2] and Md. Nazrul Islam Siddique[3]

Abstract

This paper describes the evaluation of discharge co-efficient of broad-crested masonry weirs being used as a flow measuring device. Co-efficient of discharge (C) is the dominant parameter which greatly influence on accurate flow measurement. A study was undertaken in a glass-sided laboratory tilting flume with a view to find out the discharge co-efficient of broad-crested masonry weirs using usual discharge formulae and to evaluate the performance of weir with respect to upstream and downstream slopes and their crest height. Higher co-efficients were found in downstream slope conditions than those in upstream slope conditions. The co-efficient of discharge was found to be more realistic in downstream sloping condition with crest width of smaller rather than wider crest. Higher regression co-efficient between head and discharge were found which implies that a good correlation between head and discharge was existed.

[Key Words : Broad-Crested Weir, Co-efficient of Discharge, Tilting Flume]

Introduction

Weir may be defined as a hydraulic structure used to determine the flow passing over it from measured depth above a weir crest. Generally, timber and masonry dams having various shapes of section, reservoir overflows etc. may be described as weirs. Weirs are broadly two types : sharp-crested and broad-crested. The broad-crested weirs are generally constructed in an open channel for measuring flow in a desired flow condition. Different shapes of weirs provided in the channel to minimize or increase afflux, to reduce head loss, to reduce energy dissipation, to increase co-efficient of discharge and to reduce flashing out of accumulated sediment behind a barrage, regulator, sluice, spillway etc (Hossain,1995). Commonly, rectangular, trapezoidal and parabolic weirs are designed to use in these structures. A study was carried out in a glass-sided tilting flume at Hydraulic Laboratory of River Research Institute (RRI), Faridpur with a view to find out the discharge co-efficient of broad-crested masonry weirs as shown in Figure 1 using usual discharge formulae ($Q=CLH^{3/2}$) and to evaluate the performance of weir with respect to upstream and downstream slopes and their crest height .

Available formulas are certainly inadequate to compute discharge for special type of weirs. The co-efficient of discharge 'C' varies with the conditions changes.

[1]Scientific Officer, RRI, Faridpur; [2]Senior Scientific Officer, RRI, Faridpur
[3]Principal Scientific Officer (In Charge), RRI, Faridpur

Background of the study

Hydraulic structures like culverts, sluices, spillways and regulators are constructed in different rivers, rivulets and canals to facilitate optimum water use in irrigation and drainage works. Sometimes it is necessary to know that how much water will have to be spilled over these structures. Also these structures experience a substantial sedimentation and scouring problems. In order to minimize these problems, a compromise between flow and dimensions of the structures are necessary. Flow passing through the structures are measured by constructing masonry weirs across the rivers, rivulets or canals. The weirs should be designed so that the discharge co-efficient (C) is high vis-a-vis the upstream afflux, downstream scour and sedimentation at the entrance.

Generally, experimental results on broad-crested weirs with vertical and steeper upstream slopes are available in text books. This work was undertaken to find out such configurations of masonry works which can work on the problems mentioned above also can be used as a good device for measuring the flowing discharge through these structures. In this study, different types of masonry broad-crested weirs were constructed based on crest height and slope of upstream and downstream faces.

Objectives of the study

The objectives of the study are :

- to find out the discharge co-efficient (C) of broad-crested masonry weirs using usual discharge formulae ($Q=CLH^{3/2}$) and
- to evaluate the performance of weir with respect to upstream and downstream slopes and their crest height.

Description of the flume

The flume is a self contained unit and provision has been made both for conventional sump return system and also for continuous circulation (Figure 2).

The bed of the channel is metallic and can conveniently be drilled for pressure tapping or for other purposes, if necessary. A tailgate is provided at the downstream end of the channel for depth control in the channel.

The channel is fitted with double jacking system from the center pivot point. The center distance between the jacks and pivot point is 7.98 m. The jacks are mechanically connected and manually operated. The tilting of the flume is done by turning the wheel at the pivot point. The direction of rotation of the flume is same as the direction of rotation of the wheel. One turn of the wheel will cause a rise of 0.4 mm in one side and a fall of 0.4 mm on the either side of the pivot point. One complete turn of the wheel in the anticlockwise direction will produce a normal (positive) slope of 0.000051 from horizontal. Similarly, one complete turn of the wheel in the clockwise direction will give a reverse (negative) slope of 0.000051 from horizontal.

The channel and the pipe network is designed to circulate a total flow of 0.34 m^3/s through two centrifugal pumps. The circulation of water in the channel is accomplished by a suitable designed two pipe lines system. The internal diameter of two pipes are 254 mm and 305 mm respectively. The pipe lines are fitted with control valves at the inlet (pump end) and outlet (pipe end). The flow through the pipe is measured by orifice meter fitted with each pipe line.

Weir installation

Masonry broad-crested weirs with different upstream and downstream conditions were constructed in the laboratory and placed in the bed of the tilting flume. The weirs were placed at the downstream of the flume in such way that steady uniform flow is ensured at the upstream. The downstream tail gate was fitted with the rotating wheel to ensure the free fall at the downstream end.

The flume width was reduced by 92 mm providing 2.1 m long and 46 mm thick timber plate in both sides covering overall depth at the weir section in order to prevent the breaking of side glass. The joint between timber plate and the weir filled with pooting. The timber plate was extended towards upstream until minimizing end contraction at the upstream of the weir to acceptable limit. Some arrangements were made with baffles (perforated bricks) at the upstream of the measured section to dissipate excess energy due to incoming turbulent flow from the delivery pipe ultimately to distribute uniform flow across the flume.

During test operation, required head was given in a point gauge fitted with movable rail. Required head was ensured by operating controlling valves at the upstream and the corresponding discharge was measured from the fitted mercury manometer.

Calibration of discharge co-effiecient (C)

The weir co-efficient varies as a function of the total water head above the weir crest and also as a function of the upstream and downstream faces (Horton,1906). The discharge co-efficient (C) for each of the head (H) corresponds to discharge (Q) was calculated from the well known equation;

$$\text{i.e. } C = \frac{Q}{LH^{3/2}} \quad \text{-------------------------} \ (1)$$

For each test, measured discharge (Q) are plotted against known heads (H) and standard Q-H relationships were established by least square method. From these relationships, single intercept and exponent were obtained for each relationship. Here, the intercept stands for co-efficient of discharge (C). These co-efficients were than compared with individual 'C' values obtained from standard equation (1). Percentage of error in calculation of discharge were obtained by $\dfrac{Q_{obs} - Q_{cal}}{Q_{obs}}$.

The variation of 'Q' values for different sloping face with different crest width were also investigated and compared with each other. From the established relationships, one can easily choose the desired value of 'C' for design of hydraulic structures like sluice, regulator, rivulets or spillways. In addition, overflowing discharge during flood over roads, dams, levees etc. can be computed when these acts as a broad-crested weir by knowing the value of 'C' for a particular case.

Test runs

In this study, total water head above the weir crest, crest width, upstream and downstream conditions have been considered as variables. The head varied within the range of 91.40 mm to 335.3 mm. Whereas crest width varied between 101.6 mm to 203.2 mm for different test conditions.

A total of twenty two tests were designed and planned to determine the discharge co-efficients in different conditions. Of them, seven tests were planned for upstream face vertical, eight tests for downstream face slope and seven tests for both upstream and downstream face slope of the weir. The test runs were conducted for different slopes such as 1:1, 1:2, 1:3 and 1:4 as shown in Table 1. For each of the test, length of the weir normal to the direction of flow was kept constant while the crest width and water head were variables.

Data analysis and discussions

For each test, standard Head (H) - Discharge (Q) relationship was established. The co-efficient of discharge 'C' was computed from the single plausible curvilinear relationship between H and Q. Runwise H-Q relationships with test condition are presented in Table1.

Table 1: H-Q Relationships

Run	Test Condition		Crest Width (mm)	H-Q Relationship	'C' Value	Regression Co-efficient (R^2)
	Upstream Slope	Downstream slope				
BCW01	Vertical	1:1	101.6	$Q=2.44\,L\,H^{1.58}$	2.44	0.90
BCW02	-Do-	1:2	101.6	$Q=2.44\,L\,H^{1.63}$	2.44	0.93
BCW03	-Do-	1:3	101.6	$Q=2.54\,L\,H^{1.62}$	2.54	0.92
BCW04	-Do-	1:4	101.6	$Q=2.75\,L\,H^{1.70}$	2.75	0.93
BCW05	-Do-	1:2	203.2	$Q=2.77\,L\,H^{1.74}$	2.77	0.93
BCW06	-Do-	1:3	203.2	$Q=2.99\,L\,H^{1.82}$	2.99	0.97
BCW07	-Do-	1:4	203.2	$Q=2.61\,L\,H^{1.70}$	2.61	0.98
BCW08	1:1	Vertical	101.6	$Q=2.46\,L\,H^{1.57}$	2.46	0.94
BCW09	1:2	-Do-	101.6	$Q=2.27\,L\,H^{1.50}$	2.27	0.94
BCW10	1:3	-Do-	101.6	$Q=2.30\,L\,H^{1.51}$	2.30	0.93
BCW11	1:4	-Do-	101.6	$Q=2.83\,L\,H^{1.69}$	2.83	0.92
BCW12	1:1	-Do-	203.2	$Q=1.88\,L\,H^{1.54}$	1.88	0.95
BCW13	1:2	-Do-	203.2	$Q=2.57\,L\,H^{1.62}$	2.57	0.95
BCW14	1:3	-Do-	203.2	$Q=2.40\,L\,H^{1.58}$	2.40	0.94
BCW15	1:4	-Do-	203.2	$Q=2.14\,L\,H^{1.50}$	2.14	0.96
BCW16	1:1	1:2	203.2	$Q=2.54\,L\,H^{1.64}$	2.54	0.95
BCW17	1:3	1:2	203.2	$Q=3.20\,L\,H^{1.82}$	3.20	0.93
BCW18	1:4	1:2	203.2	$Q=2.84\,L\,H^{1.68}$	2.84	0.96
BCW19	1:2	1:4	101.6	$Q=3.01\,L\,H^{1.73}$	3.01	0.96
BCW20	1:1	1:4	203.2	$Q=3.04\,L\,H^{1.77}$	3.04	0.95
BCW21	1:2	1:4	203.2	$Q=2.90\,L\,H^{1.82}$	2.90	0.92
BCW22	1:3	1:4	203.2	$Q=2.58\,L\,H^{1.65}$	2.58	0.94

BCW - Broad-Crested Weir

The discharge against known head was calculated from H-Q relationships in different test conditions and then the calculated discharge values were compared with observed values and errors in discharge were computed which are shown in Table 2.

Table 2: Errors in calculation of Discharge (Q) and Co-efficient of Discharge (C)

Run	Test Condition		Crest Width (mm)	Percent error in Discharge	Percent error in Co-efficient of Discharge
	Upstream Slope	Downstream slope			
BCW01	Vertical	1:1	101.6	5.49 to 11.60	6.13
BCW02	-Do-	1:2	101.6	4.87 to 6.67	11.17
BCW03	-Do-	1:3	101.6	4.28 to 8.79	11.79
BCW04	-Do-	1:4	101.6	4.97 to 10.79	24.02
BCW05	-Do-	1:2	203.2	8.66 to 10.88	24.59
BCW06	-Do-	1:3	203.2	7.23 to 26.37	44.06
BCW07	-Do-	1:4	203.2	31.92	25.75
BCW08	1:1	Vertical	101.6	11.43 to 28.7	4.08
BCW09	1:2	-Do-	101.6	4.34 to 4.84	4.84
BCW10	1:3	-Do-	101.6	4.94 to 6.84	3.58
BCW11	1:4	-Do-	101.6	3.34 to 9.37	26.58
BCW12	1:1	-Do-	203.2	33.41	26.72
BCW13	1:2	-Do-	203.2	4.35 to 7.05	10.11
BCW14	1:3	-Do-	203.2	7.14 to 7.51	3.54
BCW15	1:4	-Do-	203.2	5.67 to 10.78	3.52
BCW16	1:1	1:2	203.2	4.77 to 11.38	13.41
BCW17	1:3	1:2	203.2	22.45 to 23.63	11.7
BCW18	1:4	1:2	203.2	6.66 to 7.68	15.59
BCW19	1:2	1:4	101.6	6.11 to 8.13	27.10
BCW20	1:1	1:4	203.2	5.48 to 7.06	36.92
BCW21	1:2	1:4	203.2	12.62 to 18.86	27.46
BCW22	1:3	1:4	203.2	8.50 to 26.79	9.35

It is evident from the test results that the co-efficient of discharge (C) increased with the increase in water head. In all tests, the shape of curves were similar to each other. The regression co-efficients lies between 0.90 to 0.98 which reveals that the relationships were found to be reliable. In comparing observed discharge with the computed discharge, maximum about 12% and minimum 3.34% error was found in 16 tests and rest of the tests show larger variation. Errors in 'C' values lies within 3.54 to 44 percent. From the analysis it is revealed that the head-discharge relationships were consistent with the variation in exponent. Error in discharge also increases with the increases in crest width for upstream and downstream face slope or slopes in both side. Keeping downstream slope constant, error in discharge increases with flatter slope. The co-efficient of discharge is found to be more realistic in downstream sloping condition with crest width of 101.6 mm.

Conclusions

Discharge co-efficient depends on total water head above the weir crest, upstream and downstream condition. From the test results, the following conclusions can be made:

- The co-efficient of discharge (C) is increased with the increase in water level above the crest. The co-efficients were found to be higher in downstream sloping conditions than those of upstream sloping conditions.

- A good correlation between head and discharge was found in almost all tests which implies that the test results could easily be used to calculate the unknown parameters against the known value of discharge or head in a similar condition .

- The co-efficient of discharge was found to be more realistic in downstream sloping condition with crest width of 4 inches rather than wider crest.

- Based on the calibrated weirs, large size weirs can be designed to use in barrage, sluices, regulators, spillways or any other irrigation, drainage and hydraulic structures

Recommendations

The fluctuation in water level at the upstream of the weir was found during the test run at higher heads which could be minimized by increasing the length of the flume in upstream direction or measure water level in a stilling basin attached to the section.

During the test runs with higher discharge, the mercury in the differential manometer fitted with the circular pipe fluctuates frequently which results inaccuracies in discharge calculation. These inacuracy could be minimized by connecting a tube of reduced section.

The errors in calculation of discharge as well as co-efficient of discharge is influenced by constant flume width and depth. In this study water surface slope was kept constant. A good calibration can be encapsulated with the variable flume size and water surface slope.

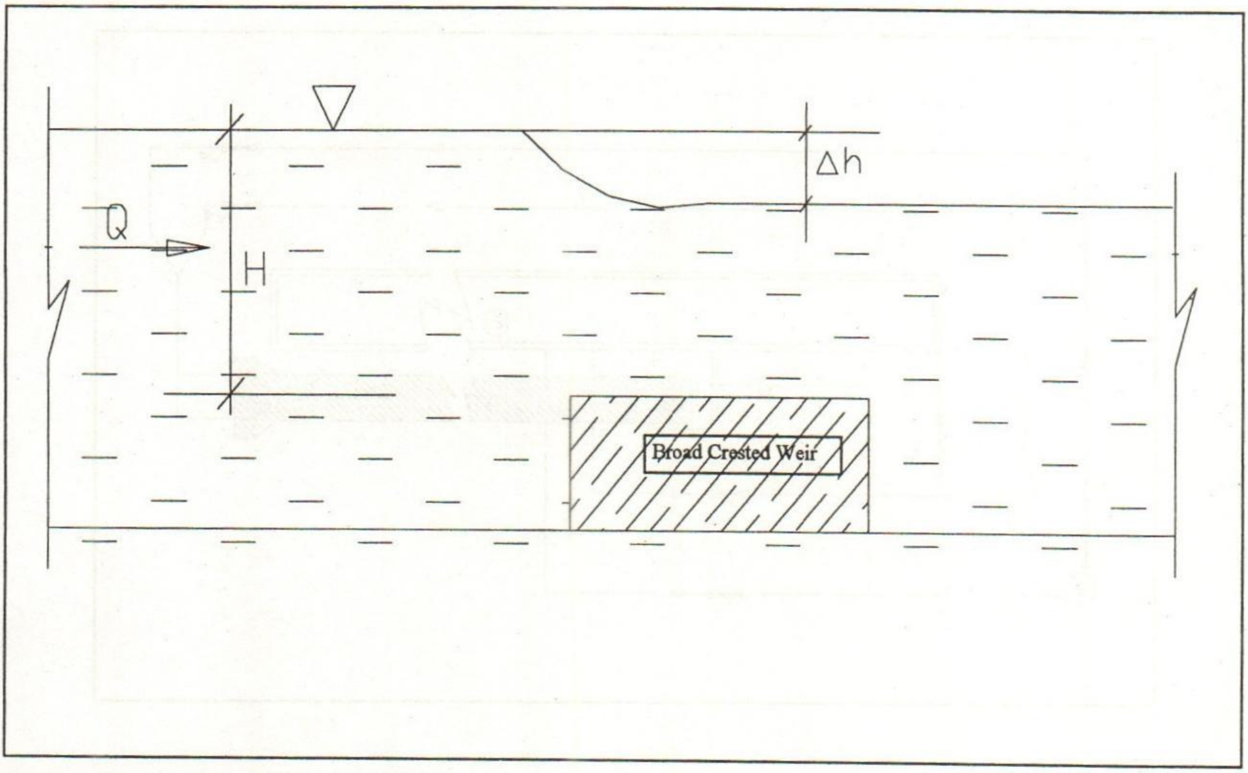
Acknowledgements

The authors acknowledge their sincere gratitude and indebtedness to Dr. M. R. Kabir, Associate Professor, Department of Water Resources Engineering, Bangladesh University of Engineering and Technology (BUET) for his guidance, co-operation and valuable suggestions and comments extended to this study. The services given by RRI officers' and staff are gratefully acknowledged.

References

Horton, R.E. (1906) **Weir Experiments**, Co-efficients and Formulas, US Geological Survey, Washington D.C., USA.

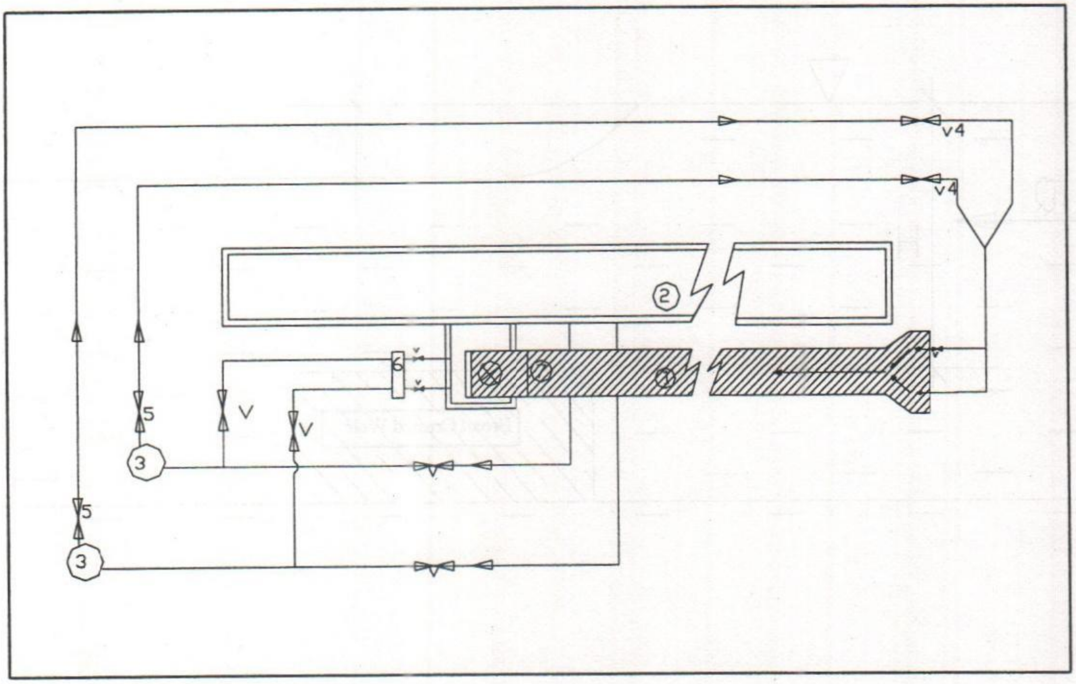
Hussain, S.M. (1995) **Hydraulic Performance of Broad Crested Weirs of Different Shapes (A Case Study)**, Technical Journal of River Research Institute, Vol. 02, No. 01, January 1995, Faridpur.



Legend

Q Discharge
H Head Above the Weir Crest
Δh Head Loss

Figure 1 : Definition Sketch of a Broad Crested Masonry Weir



Legend

1 Glass Sided Flow Channel
2 Sump Tank
3 Centrifugal Pump
4 Control Valve at the Pipe End
5 Control Valve at the Pump End
6 Manifold
7 Tale Gate

Figure 2 : Experimental Setup

SIMULATION OF MORPHOLOGICAL PROCESS IN SCALE MODEL

Pintu Kanungoe[1]

Abstract

Louis Jerome Fargue was the first to build movable bed river model in 1875. Since then growing insight into the phenomena involved has given a sounder basis for this type of model. However, due to various practical reasons it is not an easy task to conduct such model study which does not yield any error. A sound understanding of the sediment transport in alluvial rivers and morphological phenomena can help a lot to reduce errors and in interpretation of the test results. In this regard it is important to determine the scale conditions required to reproduce all the relevant processes namely water flow, sediment transport and bed topography changes. It may be found that these scale conditions are not same and as a consequence complicacy arises in reproducing the involved processes simultaneously. Therefore, an optimal solution should be sought to satisfy all the conditions required to reproduce morphological process. The other aspect of the morphological model is that it is generally too rough and there is almost no possibility to satisfy the roughness condition. It may lead either to accept a geometric scale distortion or to provide an additional longitudinal slope in advance. For simplicity, this contribution is restricted to the reproduction of morphological process in a straight section of alluvial river with relatively stable sloping bank. The sediment transport here applies to bed load transport only.

Introduction

Reproduction of morphological process in scale model requires correct simulation of flow pattern, sediment transport and bed topography simultaneously. Similarity with respect to sediment transport is of immense importance. Attempts can be made to fulfil all conditions at the same time to reproduce the three different relevant processes. But in doing so conflicting scale conditions may arise which decrease the reliability of scale model investigations more. Therefore, it is necessary to find ways and means in order to have a reliable solution. The simplified sediment transport equation is:

$s = m\,V^n$ .... (1)

---

[1]Senior Scientific Officer (In Charge), RRI, Faridpur

The above equation indicates that correct reproduction of the power n is important in morphological model.The equation of continuity for the sediment under the assumption that there are only small changes in the suspended load is as follows:

dz/dt + ds/dx = 0 ....(2)

From (1) and (2) it can be written as:

dz/dt = - mn dV/dx ....(3)

where dz/dt is change in bed level with time and dV/dx is velocity gradient. From equation (3) it can be concluded that reproduction of erosion and deposition process is possible when both the velocity gradient and n are reproduced correctly. In case of rivers with fine sand and where light weight material is not used as model bed material, similarity requirement for sediment transport leads to scale the velocity according to the *ideal velocity scale* (V_{ri}). Usually this ideal velocity scale is different from the velocity scale obtained from the Froude condition (V_{rF}). Generally $V_{ri} < V_{rF}$ (necessary for correct sediment transport) and it results in relatively large flow velocity in the model. Hence, the selection of V_{ri} leads to errors in the water levels. However, the problem is not serious if the velocity head is small compared to the depth in the model. But when the problem is serious, the scale effects for the water level can be compensated by tilting the model.

A morphological model should be undistorted to satisfy both scale conditions for flow hydrodynamics and morphology. This is possible when Chezy's co-efficient in the model (C_m) and Chezys co-efficient in the prototype (C_p) are equal. But when bed forms are more pronounced in model than in prototype, C_m is found appreciably less than C_p. The roughness of the movable bed model cannot be influenced like in fixed bed models. Hence, model has to be distorted to satisfy the roughness condition.

The facts outlined above and some practical aspects of morphological model are elaborated hereafter to overcome different problems envisaged during execution of such model study.

Selection of velocity scale

The velocity scale (V_r) according to Froude condition is

$V_r = H_r^{0.5}$ .... (4)

Where, H_r stands for depth scale.

The use of this velocity scale in morphological model is possible only when $H_r = D_r$. It means if the prototype bed material consists of coarse sand or gravel and can be reproduced in the model in depth scale provided that model bed material is not too fine, the Froudian velocity scale can be selected.

In case of sand bed rivers for which $H_r = D_r$ is not possible, the model velocity scaled according to Froude condition may not be sufficient enough to reproduce the sediment transport in the model for a particular model bed material. In that case it is important to scale the sediment transport correctly. The exact form of sediment transport equation is not known yet. But the dimensionless sediment transport can be written as a unique function of shields parameter (Bijker et. al. 1957 and Klaassen,1997) :

$$s/\sqrt{g\Delta D^3} = f(\mu Hi/\Delta D) \text{ ....(5)}$$

If it is assumed that

$$(\mu Hi/\Delta D)_r = 1 \text{ ....(6)}$$

it automatically leads to

$$\{s/\sqrt{(g\Delta D^3)}\}_r = 1 \text{ ....(7)}$$

From equation (6), $(\Delta D)_r = (\mu Hi)_r$

Using the Chezy equation, $V = C(Hi)^{1/2}$

$$(\Delta D)_r = (\mu V^2 C^{-2})_r$$

i,e, $V_r^2 = (c^2 \Delta D \mu^{-1})_r$ ....(8)

Equation (7) can be written as

$s_r = \Delta_r^{1/2} D_r^{3/2}$ .... (9)

Equation (8) is referred to as the condition for *ideal velocity scale* (Jansen et. al., 1979) because similarity with respect to the bed topography can only be obtained from this condition.

From equation (9) it can be seen that s_r is constant in space and time, since D and Δ are constant in time and space.

The morphological time scale can be written as (Matin,1995) :

$(t_m)_r = H_r\, L_r/s_r$ ....(10)

This means that the morphological time scale is constant in time and space.

It should be noted here that morphological scale model may not be distorted which means $C_r = 1$. Now, if in the model same sediment is used as in the prototype and the influence of the ripple factor (m) is considered negligible the equation (8) reduces to

$$V_r^2 = D_r$$

Therefore, if the above condition does not satisfy in the model, a deviation from the Froude condition is inevitable. But such a deviation from the Froude condition is acceptable as long as F_m does not exceed about 0.5 (G.J.Klaassen, 1997). It is important to note that $V_r^2 < H_r$ results in a relatively too steep longitudinal slope which creates problems in reproduction of correct water level. However, attempts can be made to overcome the problems by giving an additional slope by tilting the model.

The other limitation associated with the selection of the so called *ideal velocity scale* is that in cases with high Shields value i,e, large water depth, it results in high sediment rates in the model. In such case a specific sediment transport formula may provide an accurate description of the sediment transport in the prototype as well as in the model. An example for the Engelund and Hansen sediment Transport formula is given hereafter. The sediment transport formula according to Engelund and Hansen (1967) reads as:

$s = 0.05/(1\text{-}\varepsilon)^* (C^2/g) * (Hi/\,\Delta D)^{2.5} * \sqrt{g\Delta D^3}$ ....(11)

From Chezy equation "Hi" of equation (11) can be replaced by $(V/C)^2$. From equation (11) it can be seen that sediment transport is a power function of velocity. It can be accepted from the equation (11) that

$$s/\sqrt{(g\Delta D^3)} = K^* C^2/g^* V^5{}^*(C^2\Delta D)^{-5/2} \text{ .... } (12)$$

Where, $K = 0.05/(1-\varepsilon)$

The equation 12 may be rearranged as

$$V_r^5 = C_r^3 \Delta_r^2 D_r s_r \quad \text{..................................} (13)$$

Therefore, another velocity scale is found which allows for deviations from the *ideal velocity scale*. But, since in this approach the assumption is that both the scale of the dimensionless sediment transport and Shields parameter should be unity is dropped and the power n of the simplified sediment transport formula is a function of the Shields parameter, n is no more reproduced correctly in the model. However, the selection of the sediment transport scale provides more freedom for the selection of the velocity scale.

Similarly the Meyer-Peter and Mueller formula and other sediment transport formula's can be used based on the knowledge of their appropriate application.

Use of light weight material in the model

It can be seen from the equation (8) that increase in the *ideal velocity scale* upto the extent of Froudian velocity scale is possible by increasing D_r. It can be realized by using a lighter model bed material than the sediment in the prototype (D_r>1). Therefore, the condition that $H_r = D_r$ is no more required to be fulfilled. The disadvantages in the use of light weight materials are:

a) It is not easily available

b) Expensive

c) Generally, it is uniform and cannot be used to study the river problems where sorting is important

Therefore, in the morphological model study the scope of the use of light weight material is limited.

Roughness condition

In morphological model study it is a prerequisite that the following roughness condition is satisfied properly.

$C_r^2 = L_r/H_r$ where, $C_r = C_p/C_m$

If the above condition is not satisfied the consequences are as follows:

a) $C_r^2 > L_r/H_r$ indicates a too rough model where water levels and curvature of the flow lines are too large.

b) $C_r^2 < L_r/H_r$ indicates a too smooth model where water levels and curvature of the flow lines are too small.

A good reproduction of morphological process in scale model is possible only when $C_r = 1$ i,e, the model is undistorted. But for a sand-bed river and sand-bed model it is almost impossible to realise $C_r = 1$. This is due the fact that in this case roughness both in the model and the prototype is a function of the inherent formation of bedforms and the bedforms are more pronounced in the model than in the prototype. Hence usually $C_m < C_p$ and geometric scale distortion is required to satisfy the roughness condition which leads to scale effect. Figure-2 shows the incorrect reproduction of water level in the model compared to the prototype situatuion due to the fact that roughness condition is not satisfied. But it is possible to eliminate scale effect by increasing the slope of the model during calibration of the model. Now-a-days there are 2D mathematical models of river morphology which can be used to study the scale effect and to take it into account during the interpretation of the model results.

Increase of the model bed level slope

In morphological model deviation from the Froude velocity scale i,e, the selection of $V_r^2 < H_r$ leads to errors in the water levels.From the Chezy equation the following scale relation can be derived

$i_r = V_r^2 C_r^{-2} H_r^{-1}$ ....(14)

If the roughness condition is satisfied Equation (14) becomes

$i_r = V_r^2 L_r^{-1}$ ....(15)

The slope of the model is equal to the difference in energy head over the distance L. For moderate Froude numbers it implies,

$i = \Delta H/L$

Now, incorporating the deviation from the Froude velocity scale it can be written from the equation (15):

$V_r^2 = (\Delta H)_r < H_r$

$i_r < H_r/L_r$

The above relations indicate that the differences in the peizometric head are exaggerated compared to the vertical scale due to increase in the model flow velocity and the scale for the bed level slope (i_r) should be smaller than the length scale divided by the water depth scale. It means the model bed level slope should artificially be made larger than the bed level slope in the prototype i,e, the model should be tilted as shown in Figure 1.

The relation between bed level slope in the model and in the prototype is:

$i_m = ri_p$, where $r = L_r/H_r$

Therefore, the required tilting of the model slope (i_t) should amount to

$i_t = i_m - ri_p$

Figure 1 shows that if the water level in the centre of the model is correct then at the end of the model an error in water level ΔH will occur due to $V_r^2 < H_r$, which can be represented as

$\Delta H = \pm \frac{1}{2} L_m (i_m - ri_p)$

It is clear from the Figure 1 that by providing an additional slope i_t in the model such error in the water level can be diminished. But the estimation of i_t depends on the estimation of the model bed roughness. The roughness of the model bed however is difficult to estimate, because for a movable bed model this roughness is determined by not only the grain roughness, but by the form roughness as well. Therefore, the estimated model roughness may not be accurate enough because the bedform roughness is still poorly understood. Hence, the actual slope may be different from the estimated slope. In practice this limits the length of a reach that can be modelled in a movable bed model. However, some idea about the bedform roughness of the model can be obtained from either earlier experience with the proposed bed material or via special flume tests with the selected bed material in which roughness is measured for the anticipated slope and water depth in the model.

Practical aspects of morphological models

The following practical aspects of the morphological models are important to be taken into account during planning phase of the model study.

a) Reproduction of a long river reach within the limited laboratory space might result in the selection of a large length scale. Large length scale means large velocity scale according to Froude condition and small flow velocity in the model. Since this velocity is not sufficient enough to reproduce the sediment transport rates, a deviation from the Froude velocity scale is essential. As a consequence, the ratio of the velocity head and the water depth in model is too large compared to the prototype conditions. It leads to scale effect which can not be compensated even after tilting the model. Therefore, the river section to be reproduced in the model should be as short as possible and it can be realized by computing the upstream and downstream boundary conditions using mathematical model.

b) Lack of adequate pumping capacity may lead to accept a model discharge less than that necessary to ensure ideal velocity condition in the model. This will result in the fact that the sediment transport scale and the morphological time scale will vary for the accepted discharge. It should be considered in the interpretation of the model results.

c) In case the model is built to study the overall bed topography, a distortion of the model is acceptable within reasonable limit to satisfy the roughness condition. But for models of local scour processes non-distortion condition should be maintained strictly. This is due to the fact that the vertical accelerations caused by vortices are not correctly reproduced in the distorted model. In local scour model there is no need to satisfy the ideal velocity criterion. Rather the condition $V_m = V_c$ should be satisfied. V_c in the model can be calculated by use of the following formula(Rijn, Leo C. van, 1993).

$$V_c = 0.19\,(d_{50})^{0.1}\log(12H/3d_{90}) \quad \text{for} \quad 0.0001 \le d_{50} \le 0.0005\text{m}$$

$$V_c = 8.50\,(d_{50})^{0.6}\log(12H/3d_{90}) \quad \text{for} \quad 0.0005 < d_{50} \le 0.002\text{m}$$

in which

d_{50} = median particle diameter (m)
d_{90} = 90% particle diameter (m)

Attempts can be made to fulfil this criterion together with the Froude condition. Scale effect introduced due to non-distortion of the model can be minimised by selecting a small length scale.

d) In mobile bed models the sediment can either be fed artificially at the upstream end,or it can be recirculated. The advantage of the upstream feeding system is that the sediment transport in the model is controlled and in combination with the discharge the slope of the model is controlled as well. A disadvantage is that much labour is needed to feed the sand unless complicated automatic systems are built. The latter systems might be sensitive to breakdown, resulting in periods that no sediment is fed to the model and hence to bed disturbances.The alternative is a recirculating system, but this has the disadvantage that the sediment transport is a dependent variable and might be difficult to control (Klaassen, 1997).

Conclusion

It can be concluded from the above discussion that in order to reproduce all the active processes in morphological model a number of considerations are involved from which sometimes different and sometimes similar scale conditions are derived. When the suspended load vertical and changes in the planform over time are taken into account even more scale conditions arise which means more conflicts. Therefore, for morphological model there is no single best solution. Rather an optimal solution should be found to satisfy the conditions required for the reproduction of all relevant processes.

References

Jansen,P.Ph.(Ed.) (1994) **Principles of River Engineering**. The non-tidal alluvial river, DUM, facsimile edition of Pitman Publishing Company (1979).

Klaassen, G.J. (1997) **Scaling of sediment transport and morphological models**, Lecture Notes, Delft Hydraulics, The Netherlands.

Matin, M.A. (1995) **Scale Modelling**, Lecture Notes, Dept. of Water Resources Engineering, BUET.

Rijn, Leo C. van (1993) **Principles of sediment transport in rivers**, estuaries and coastal areas, AQUA Publications.

Struiksma,N (1986) **Scale effects on the reproduction of the overall bed topography in river models**, Symposium '86 on Scale Effects in Modelling Sediment Transport Phenomena, Toronto, Canada.

List of Symbols

H	water depth
L	length
D	grain size diameter
g	acceleration due to gravity
i	energy gradient
s	sediment transport
m	ripple factor
D	relative submerged density of sediment
e	porosity of settled bed material
C	Chezy bed roughness coefficient
V	flow velocity
n	exponent in simplified sediment transport formula

Captain Mohiuddin Zahangir College, which is the only institution for higher education in that region, is threatened due to degradable nature of the Ganges. Recently, Bangladesh Water Development Board (BWDB) has undertaken a development project for the protection of that area. This study was a part of that project. In this study revetment was used as a protective structure as it was suggested by the BWDB, Northern Zone, Rajshahi.

The Ganges river

The Ganges is one of the major rivers in Bangladesh and is also noted for its massive water discharge and huge sediment load. The length of the Ganges from its origin at Himalayas to the out fall into the Bay of Bengal measured along the Hooghly in India is about 2525 km and along the Padma-Meghna is about 2650 km of which 375 km is in Bangladesh. Among the three major rivers in Bangladesh, the Ganges River has the highest drainage basin of some 1.1 million km^2 (Barua, 1994), yet its water yield is the least of the three rivers. This is primarily due to the distribution of rainfall in the drainage basins which is higher in the eastern drainage basins and lower in the western drainage basins. Other notable characteristics of the Ganges are its wide meandering planform (Figure 1), a bed-level slope of 5×10^{-5}, and an average bed material grain size of about 0.12 mm (Khan and Barua,1994). The bank-line migrations are very erratic in nature and varies widely from bend to bend as well as from year to year. It is more sinuous in the upper reaches and less sinuous tending to straight braided pattern in the lower reaches (Hossain, 1989). Maximum daily discharge of the Ganges river recorded at Hardinge Bridge in Bangladesh is 76000 m^3/sec on September 1987 while the minimum recorded discharge at the same place is only 261 m^3/sec on March 1993. Maximum and minimum water level at Hardinge Bridge from the year 1970 to 1994 is 14.86 m PWD and 5.03 m PWD respectively (source : from data of BWDB used in this model).

An overview of revetment as a bank protective device

Revetment is a slope protective structure designed to protect the loss of soil and stabilize the slope from the action of currents and waves. It is used for the protection of the slope of the river bank, shoreline and the slope of embankment or the structure alike.

The use of revetments as a bank protective device depends on the nature of the problem area where there is a sloping bank as well as other possible measures do not seem to be fit. There are different types of revetments and they can be distinguished in several groups where the major types are rip-rap or uniform rock, concrete armouring units, regular placed stones or concrete blocks.

Revetment structures are mainly used in the guide bunds of the bridge, town protection etc. The effective functioning of it depends on many engineering and physical considerations in the design process. These are stability, flexibility, durability, possibility of inspection of failure, easy placement and repairing, overall safety etc. The best revetment is one which combines all these functions (Verhagen, 1995).

The revetment should be sufficient rough to create a zone of intensified turbulence and low velocity in the vicinity. This trends hold the high velocity flow away from the revetment rather than in contact with it with the result that the revetment toe is less apt to be undercut by scour (Vanoni, 1997). In support of this criteria there is an example of experiment on the Missouri river with rough rock revetment, however, has been that the high velocity flow previously moving adjacent to the bank moves approximately 15.24 m riverward, thus holding the area of deepest scour away from the toe of the revetment and limiting the opportunity for undercutting.

The formula proposed by Isbach (1936) [Vanoni, 1977] for the criterion of design by depositing rock in running water, modified to take into account the slope of the bank, gives results that are in line with experience.

$$W = (4.1 \cdot 10^{-5} G_s V^6)/((G_s - 1)^3 \cos^3 \varphi_1)$$

in which, W = weight of the stone in lb; G_s = specific gravity of the stone; V = the velocity and φ_1 = the angle of the pavement with the horizontal.

In consideration of risk acceptability in comparison to other river training structures, revetment occupies an advantageous position. Damage to a revetments can be accepted even with a much higher frequency. Annual damage to a revetment does not cause serious problems, if this damage is repaired regularly by a good maintenance programme. The acceptable damage level in this case is completely determined by the optimization between initial costs and maintenance costs.

Study area

A total of 3.00 km river length was of interest to protect the area - Captain Mohiuddin Jahangir College (Fig 2) and its vicinity effectively. The Captain Mohiuddin College was taken as a key point while 1.80 km in the upstream and 1.2 km in the down stream were considered for the study. A total of 600 m river width was considered in the model as the available pump capacity and model bed restrict the total width of the river for consideration. In this area river bank was almost vertical and the bed topography was irregular (BWDB data of 1996). An over all features of the study area is given in Table1.

Table 1: General features of the Study Area

No	Description		Unit	Magnitude
1	Length of the reach		meter	3000
2	Mean river width		m	3200
3	Sectional width of the river		m	600
4	Mean water depth		m	12.0
5	Maximum water level		m PWD	23.19
6	Design discharge		m^3/s	56500
7	Sectional discharge		m^3/s	20956
8	Average velocity		m/s	2.91
9	Water surface slope		-	$1.5x10^{-4}$
10	Kinematic viscosity		m^2/s	$9.5x10^{-5}$
11	Bed material size	D_{50}	mm	0.16
		D_{90}	mm	0.24

Data collection and assessment

Collection of data

An index map of the river reach showing bank line, flood embankment was collected from the BWDB. The location of the cross sections with chainage, char areas, thalweg, flow direction, area of the flow concentration etc were shown on that map. Bathymetric data of some 19 cross sections at an interval of 100 m in the problem area and 300 m interval in other areas (4.15 km in the upstream and 3.93 km in the downstream of the problem area) were supplied by BWDB. Water level and discharge data of 25 years recorded at the Hardinge bridge and water level of 4 years recorded at Panka Narayanpur were collected. Some soil samples of river bed and bank material were collected and tested for gradation and grain size analysis.

Estimation of discharge

There was no discharge measurement station at Panka-Narayanpur area. Only a station at Hardinge bridge was available in the Ganges River which is about 138 km downstream of the study area. In the assessment of discharge at Panka Narayanpur area, recorded discharge at Hardinge bridge and the water level at the study area were used. The discharge value was assessed using Manning's formula calculated by spreadsheet analysis. The discharge value for the highest flood level of +23.19 m PWD was 56500 m^3/s. The corresponding discharge for sectional model was calculated from area discharge relationship at a representative section and found as 20956 m^3/s. The discharge in the model corresponding to design discharge is 366 l/s. In case of scour simulation, model discharge was taken higher than the discharge calculated according to the Froude's condition. This higher discharge was calculated on the basis of critical velocity in the model. Critical flow velocity for sand movement in the model (V_{cr}) was calculated according to Van Rijn equation (Rijn, 1984) as given below.

$$V_{cr} = 0.19(d_{50})^{0.1} \log \frac{12H_m}{3d_{90}}$$

Where, d_{50} = mean grain diameter, H_m = water depth in model.

In general practice, model velocity is generated 1.5 to 2 times higher than the critical velocity to ensure the sediment movement and for earlier scour development. In this study model velocity was developed 1.5 times its critical velocity and the corresponding discharge was 466 l/s in the model.

Model set up

Selection of model scale

In the selection of scale there always involve compromises between a set of factors. It depends on the hydraulic modelling criteria, type of problems to be studied, available data for calibration, economy, time and the available space for the model study (Tesaker, 1986). Considering the above criteria a length scale ratio of 1:80 was selected for the present model study. In the derivation of the other scale ratios Froude's model law was used. The scale ratios for discharge and velocity were calculated as 57243 and 8.94 respectively. These scale ratios lead to obtain approximate Reynold's number of 195,000 and an average water depth of 15.0 cm, which ensured the criteria for minimum Renold's number of 3000 and the minimum water depth of 3 cm necessary for proper simulation (Montensen, 1984).

Preparation of model bed

An open air model bed having the dimension 60 m X 30 m was used for this model study. Some 14 cross sections (C/S-07 to C/S-20) covering approximately 3000 m river length and average river width 600 m was reproduced in the model (Fig 2). The model was designed and constructed in such a way that separation of flow does not occur during the model test. The model was constructed having fixed bed, 8 m in the upstream and 4 m in the downstream, and a mobile bed of 37.5 m in the middle. A standard sharp crested weir was installed at the upper end of the model bed for the measurement of the inflow water. At the end of the model bed tail gates were installed to control desired water level in the model. Three point gauges were constructed along the right bank to measure water level in different sections during model run. Provision

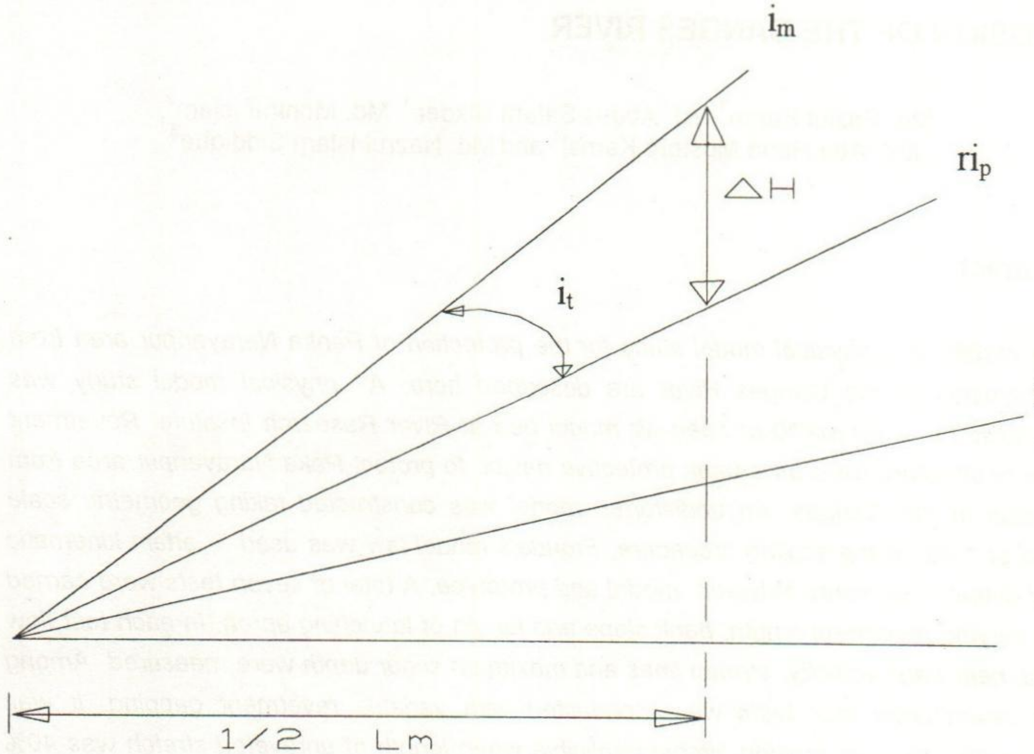


Figure - 1 : Tilting of a Model

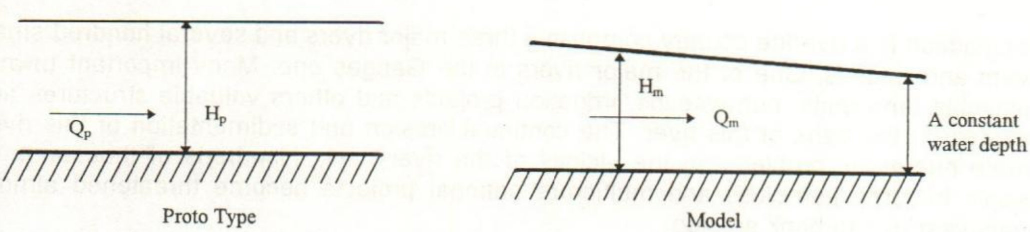


Figure - 2 : Error in the reproduction of water level due to non-fulfilment of roughness condition.

USE OF REVETMENT WITH GAPPING APPROACH: A CASE STUDY FOR THE PROTECTION OF PANKA NARAYANPUR AREA FROM THE EROSION OF THE GANGES RIVER

Md. Fazlul Karim[1],Md. Abdus Salam Sikder [1], Md. Monirul Islam[2], Md. Abu Hena Mostofa Kamal[1] and Md. Nazrul Islam Siddique[3]

Abstract

The results of a physical model study for the protection of Panka Narayanpur area from the erosion of the Ganges River are described here. A physical model study was conducted in a 60 mX30 m open air model bed at River Research Institute. Revetment type of structure used as a bank protective device to protect Paka Narayanpur area from erosion of the Ganges. An undistorted model was constructed taking geometric scale ratio of 1:80. In the scaling procedure, Froude's model law was used to attain kinematic and dynamic similarity between model and prototype. A total of seven tests were carried out varying revetment length, bank slope and length of launching apron. In each test flow field, near bank velocity, stream lines and maximum scour depth were measured. Among the seven tests four tests were conducted with variable revetment gapping. It was observed that bank erosion almost negligible when length of unreveted stretch was 40% of reveted stretch. On the basis of test results alternate gapping approach was recommended to be used with a lower value than 40% for the bank protection as a pilot case for a small river but not for a giant river like the Ganges.

Introduction

Bangladesh is a riverine country comprising three major rivers and several hundred small rivers and rivulets. One of the major rivers is the Ganges one. Many important towns, cultivable land units, homesteads, irrigation projects and others valuable structures are situated on the bank of this river. The continual erosion and sedimentation of this river create numerous problems in the vicinity of the river bank. Hundreds of thousands of people become homeless and multicrore national projects become threatened almost every year due to bank erosion.

Paka and Narayanpur, which are situated on the left bank of the Ganges, are the unions under Sadar Thana in Chanpai Nowabganj district. These are the most vulnerable area in which major part of the area is now washed out into the Ganges. One of the country's ancient high school at Paka Narayanpur is now disappeared due to the shifting of river bank.

---

[1]Scientific Officer, RRI, Faridpur; [2]Senior Scientific Officer, RRI, Faridpur
[3]Principal Scientific Officer (In Charge), RRI, Faridpur

was made for watering the model bed from the downstream prior to the model run to minimize model bed erosion.

Operation of model

The water supplied to the model from a constant head sump. The water was passed over a 1.78 m long sharp crested weir. A stilling basin was constructed immediately after the weir to dissipate energy and to create uniform flow over the model bed. The water then passed over the sand bed and discharged over the radial tailgate, and returned to the sump. A well gauge was installed at the upper side of the weir to measure the water depth flowing over the weir. Then the model discharge was measured using the Rehbock's formula as follows:

$$Q = (0.403 + 0.053\ \frac{h_w}{p})x\, b\sqrt{2g}\,[(h_w + \frac{V_a^2}{2g})^{1.5} - (\frac{V_a^2}{2g})^{1.5}\ ]$$

Where, Q = discharge in m^3/s; P = Height of the weir in m; h_w = Head over the weir in m; b = length of the weir in m; V_a =Velocity of approach in m/s; g = Acceleration due to gravity m/s^2.

At the beginning of each test run the model bed was remoulded such that same bathymetric condition exists. The discharge for each test was kept same so that the outcome of the model study represent a quantitative results. During the test run, velocity was measured along the cross-section with an interval of 25 cm and 10 cm along the bankline in the model. The water surface and bed surface readings were taken at half an hour interval at selected points. The recording of bed level was continued until the bed reading changed so slowly with time that it was difficult to read a change.

Description of the test runs

A total of four tests with discontinuous revetment have been executed in this study. In all the tests two layers of blocks were placed in the sloping surface while three layers of blocks were placed in the launching apron. In the first test, two unreveted stretches were selected randomly. Length of each unreveted stretch was 100 m. The unreveted stretches were located between C/S-11 & C/S-12 and C/S-16 & C/S-17 respectively. In the other three test runs alternate gapping revetment approach were provided (Figure 2). In this figure gap ratio was provided 40%. The length of unreveted portion was determined as some percentage of the length of the immediate upstream reveted portion. Three different gap ratios were used in the second test. At the upstream the gap ratio was 40%, at the middle it was 50% and at the downstream it was 100% of reveted stretch. In the third and fourth test fixed gap ratio of 40% was used throughout the length. In the third test blocks were placed on the bank while in the fourth test blocks were placed on the slope keeping the reveted and unreveted top levels same. In all cases the length of reveted stretch was 50 m. A brief descriptions of the test runs with discontinuous revetment are given in Table - 2 :

Table-2 : Descriptions of the test runs and the results

Test No	Revetment Condition	% Gap	Length of Each Reveted Stretch	Revetment Condition
1	Two Unreveted Stretches	-	-	8-13.7
2	Alternate Gapping	U/S-40% Middle-50% D/S-100%	50 M	7.5-14.0
3	Alternate Gapping	50%	50 M	7.5-14.0
4	Alternate Gapping	40%	50 M	8.0-14.0

Results And discussions

In the first test severe bank erosion was observed at both the unreveted stretches. This is due to the fact that unreveted stretch having a long length. Due to large gapping, effect on reveted portion was negligible at the middle of the unreveted portion. In the second test variable bank erosion was also observed at all the unreveted stretches depending on the gapping ratio. In the upstream less erosion was observed while in the downstream greater erosion was observed. The variation of bank erosion for larger percentage of gapping is shown in Figure 3. In the third test small erosion was observed but situation is improved than preceding test. In the fourth test with 40% gapping ratio least erosion was observed among all the tests and to be tolerable. On the basis of test results it is concluded that the gapping ratio of a unreveted streach to the immediate upstream reveted streach is one of the most important factor to save the unreveted bank, eventually the success of gapping approach. Again the results show that bank erosion is different at different places for the same gapping ratio.This is due to different bed topography and bank alignment (Fig 3).

Conclusions and recommendations

In this study it was observed that unreveted stretches seemed to be stable against bank erosion when unreveted stretch is 40% of the reveted stretch of length 50 m. In all cases near bank velocity was also substantially reduced.

Optimum length for each strip of revetment and succeeding gap must be determined through the physical model study for a specific river and its reaches considering representative hydrological, morphological and hydraulic data.

In this study it was observed that revetment with alternate gapping approach is technically viable to control bank erosion effectively. But before application to the major rivers through investigations are necessary with different bank alignment, bathymetry, flow direction, discharge and bed & bank materials.

It is recommended on the basis of test results that this alternate gapping approach may be used primarily for the bank protection of a small river as a pilot case but not for a giant river like the Ganges. However, in this case gaping ratio should be considerably less than 40% ensuring a good safety margin to tackle the practical situation of the river.

Acknowledgements

The authors would like to thank all the officers and staff of RRI those were involved in this study. The financial support provided by the BWDB, Northern Zone, Rajshahi is gratefully acknowledged.

References

Barua, D.K. **(1994) On the environmental controls of Bangladesh river systems**, Asia Pacific Journal on environment and Development, Bangladesh Unnyan Parisad, 1(1), 81-98.

Hossain, M.M. **(1989) Geometric Characteristic of the Padma River upto Brahmaputra confluence**, Final Report, R 02/89, IFCDR, BUET, Dhaka.

Khan, Z.H. and Barua D.K.**(1994)**, Seasonal Variation of Certain Hydraulic Parameters of the Ganges River, Proceedings of the Technical Sessions, 39th Annual Convention of IEB, Chittagong, Bangladesh.

Montensen, P. **(1984) Physical Modelling Of Open channel Flow**, Lecture Notes, Danish Hydraulic Institute.

Verhagen, H. J. (1995) **Design and Construction of Dikes for Rivers and Reservoirs**, Lecture Notes for the international cours on Land Drainage,Institue for Land Reclamation and Irrigation,Wageningen,The Netherlands

Tesaker, E.(1986) **Some Aspects of Hydraulic modelling**, Lecture Notes for RRI, UNDP/DTCD Project.

Vanoni, V. A. (1977) **Sediment engineering prepared by the ASCE Task Commitee for the preparation of the mannual on sediment of the sediment committee of the Hydraulic Division.**

Rijn, Van (1984) **Sediment transport, Part III : Bed forms and alluvial roughness,** Journal of hydraulic engineering, vol.110, No. 12, December.

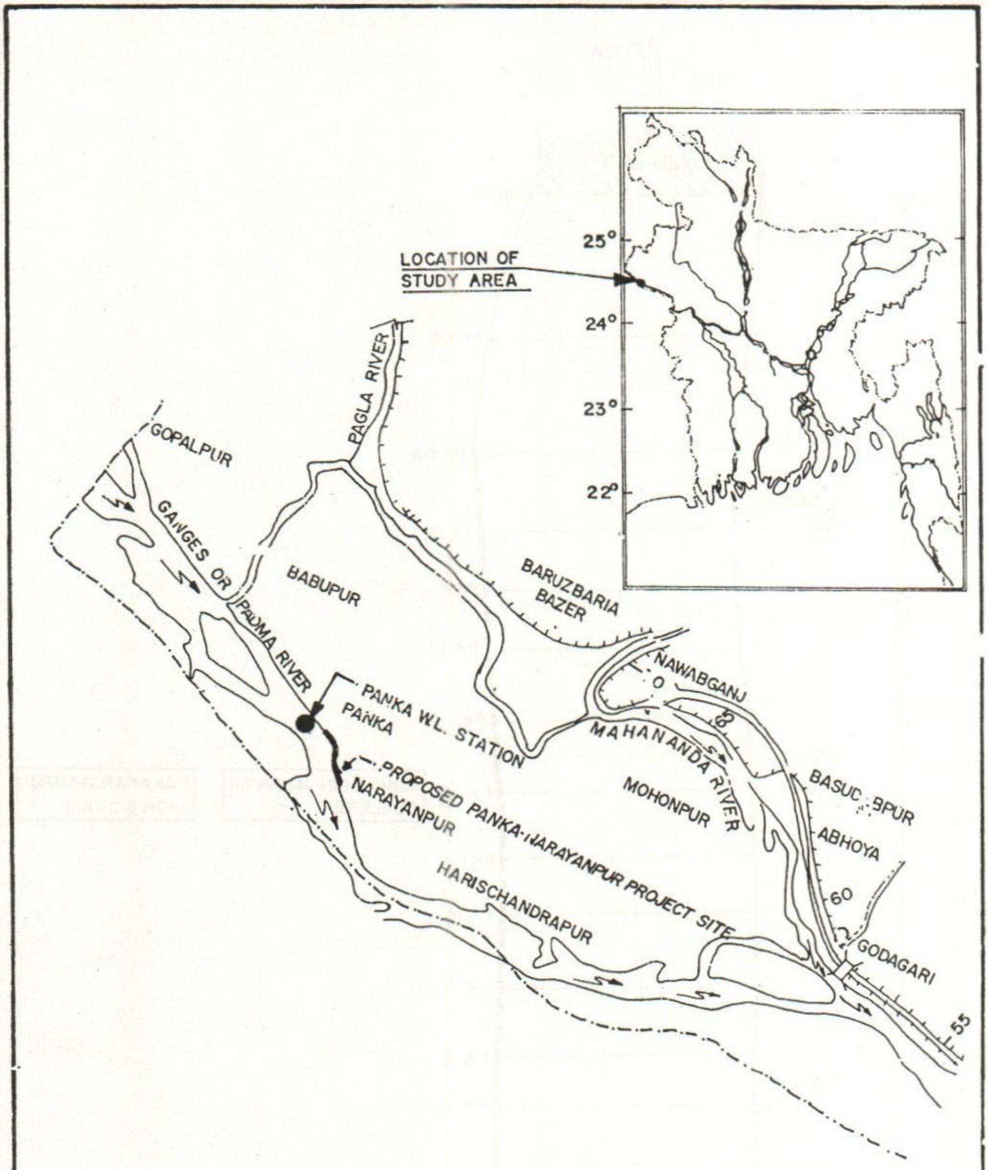


Fig- I : The study area

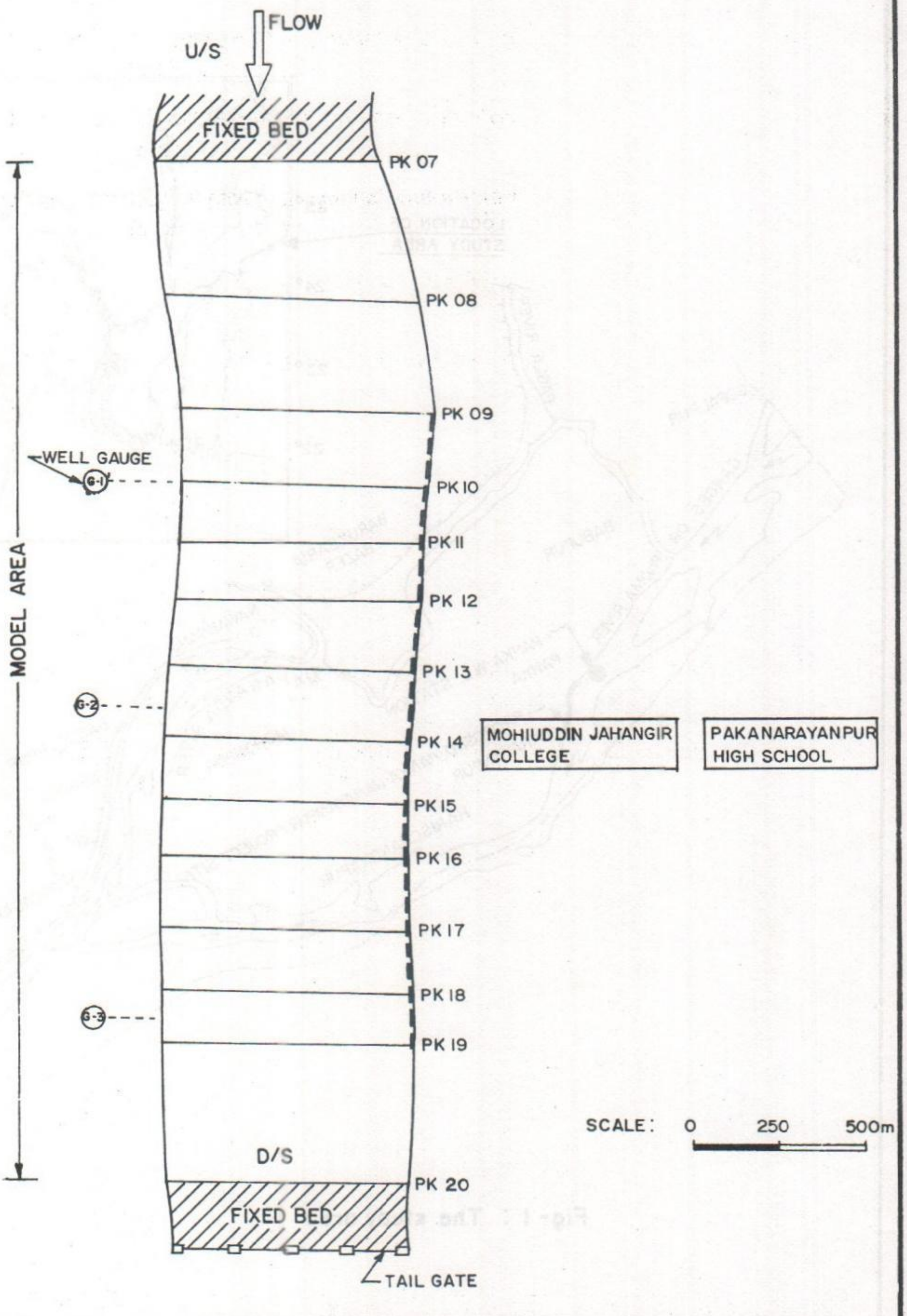
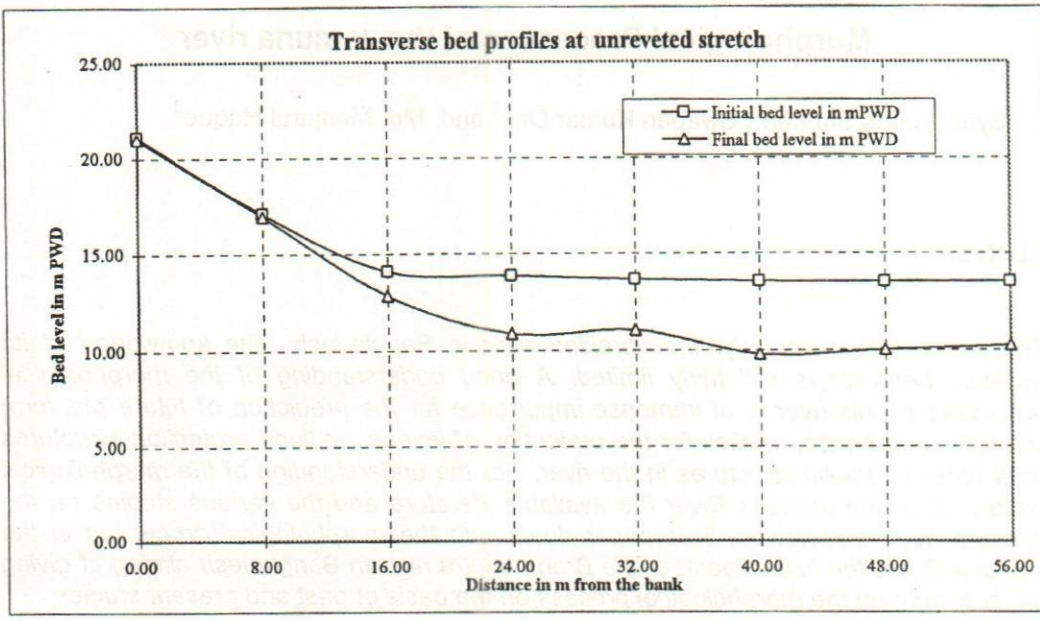
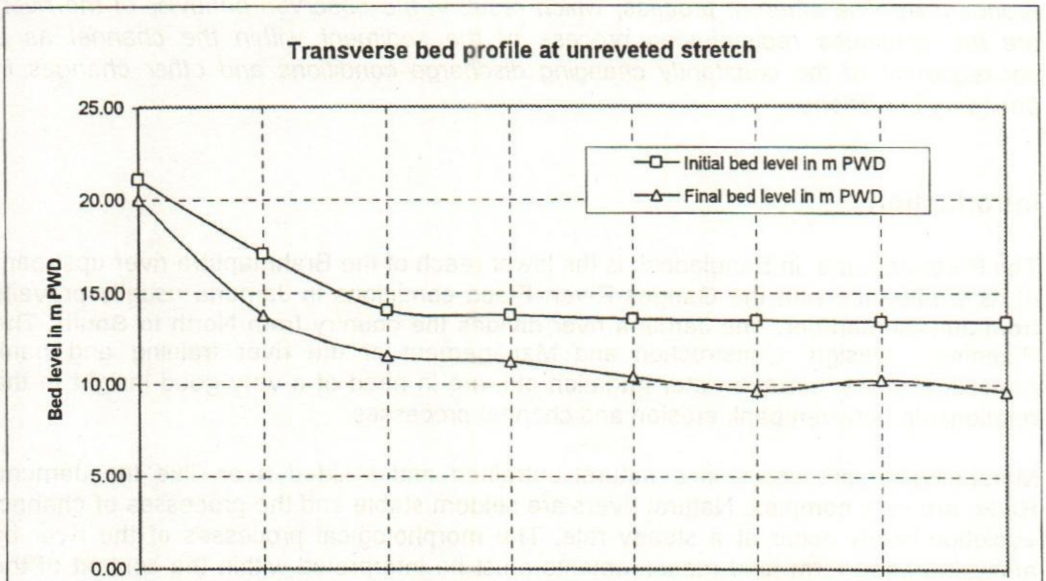


Fig: 2 : Layout plan of the model where alternate gapping revetment (40%) is shown (T12)



a) Unreveted stretch is 40% of reveted stretch



b) Unreveted stretch is 100% of reveted stretch

Fig.-3 : Transverse bed profiles for different gap ratios at CS# 16

Morphological Processes of the Jamuna river

Syed Abdus Sobhan[1], Swapan Kumar Das[2] and Md. Manjurul Haque[3]

Abstract

The Jamuna is a very dynamic braided river in Bangladesh. The knowledge of its dynamic behavior is still fairly limited. A good understanding of the morphological processes of this river is of immense importance for the prediction of future planform changes and, hence, implies for the protection of levees, or flood protection structures or of other hydraulic structures in the river. For the understanding of the morphological processes in the Jamuna River the available literature and the various studies on the Jamuna river is reviewed. This paper deals with the morphological processes in the Jamuna River, the lower reach of the Brahmaputra river in Bangladesh aiming at giving better insight on the morphological process on the basis of past and present studies.

The changes in flow direction, channel topography and planform, the occurrence of new channels and abandonment of old channels, changes of other morphological features like change of bank erosion rate, discharge dependent scour holes etc. are quite pronounced. The inherent process, which result in the observed behavior of the river, are the continuos redistribution process of the sediment within the channel as a consequence of the constantly changing discharge conditions and other changes in boundary conditions.

Introduction

The River Jamuna in Bangladesh is the lower reach of the Brahmaputra river upstream of its confluence with the Ganges River. Flood conditions in Jamuna usually prevails from July-September. The Jamuna river divides the country from North to South. The Planning, Design, Construction and Management of the river training and bank protection works, surface water irrigation etc. are in need of a very good insight in the relationship between bank erosion and channel processes.

Morphological processes in a natural untrained and braided river, like the Jamuna River, are very complex. Natural rivers are seldom stable and the processes of channel evolution rarely occur at a steady rate. The morphological processes of the river by analyzing short-term field measurements must be interpreted within the context of the overall characteristics of the river, its historic and recent development.

---

[1]Director General, RRI, Faridpur; [2]Senior Scientific Officer, RRI, Faridpur
[3]Scientific Officer, RRI, Faridpur

Characteristics of the Jamuna river

The Jamuna River is the lower part of the Brahmaputra River in Bangladesh. The Brahmaputra river originates in Tibet on the North slope of the Himalayas and drains an area of about 550,000 km^2, extending over China, Bhutan, India, and Bangladesh. Its length is about 2,740 km before meeting with the Ganges River at Aricha, of which only 240 km is in Bangladesh. The reach downstream of the offtake of the old Brahmaputra River is named the Jamuna River.

The Jamuna River is characterized by its highly variable discharges ; its yearly variation ranges from 4,000 m^3/s to 68,000 m^3/s (Khan, 1995), whereas the yearly average discharge is about 20,000 m^3/s. The peak discharges occur during the period of July-August and the lowest flow in February-March of the year. The bankfull discharge of the river is about 48,000 m^3/s. The slope of the river within Bangladesh decreases in downstream direction and is 8.5×10^{-5} where the river enters the country and 6.5×10^{-5} near the confluence with the Ganges River. The bed material sizes also decrease from the upstream towards the downstream and range from 0.22mm to 0.16 mm (d_{50}). The yearly sediment transport through the Jamuna River was estimated by various authors and in various studies in Bangladesh. The estimation of yearly average sediment transport varies from 400 to 800 Million tonnes(Mt). The recent findings was 590 Mt/year, almost in between the extremes. From this the estimated sand fraction is about 200 Mt/year (RSP, 1955a).

The Jamuna is a braided river with a braiding index that varies spatially as well as with time. The range of variation is 2 to 5 (Klaassen and Vermeer, 1988). The overall width of the river also varies spatially and temporally, from 6 to 14 km. Generally the braiding index and overall width are larger at the upstream than further downstream, probably because of the effect of higher slope and grain sizes (Klaassen and Vermeer, 1988). The river has an increseaing trend in overall width and a westward shifting tendency, especially at the upstream part of the river within Bangladesh. The widening can be attributed to an advancing alluvial fan or to the not-yet completed adaptation processes after the shift to its new course.There are different types of channel exists within the overall width of the river. The river often displays two major anabranches flowing down the right and left banks of the braided belt. In addition, Bristow (1987) proposed a classification of river channels into different orders. The entire channel is the first-order channel comprising, which in turn have smaller channels classified as third-order channel. The second-order channels have slightly different characteristics, like slightly different slope, different water and sediment-carrying capacities and as a result, they behave differently (RSP; 1996).

The shifting characteristics of the river can be divided according to the order of channel i.e. the shifting of the first-order channel differs from the second order channel. The shifting rate of the first order channel of the Jamuna River is on the average 75 to 150 m per year over the past decades. The second-order channels change continuously, large channels are abandoned, and new ones are developing in a few years only. A bank erosion rate of the second-order channels of 250 to 300 m per year is common and, in extreme cases, it can be more than 800 m/year (Klaassen and Masselink, 1992).

The bed configuration of the Jamuna River changes drastically under different flow regimes. Deposition of sediment in one location causes deepening and scour in another location. This process is associated with the continuos wandering of the thalweg from one position to another. The process of erosion and deposition is pronounced during the high flow stages and slows down during the falling stages of discharge.

Historic development

About two hundred years ago the Brahmaputra river flowed through the present course of the Old Brahmaputra River. Although much of the Bengal basin is composed of recent sediment deposited by the Ganges, Brahmaputra, Meghna rivers and their large number of tributaries and distributaries, four Pleistocene alluvial terraces exit in the basin and the Madhupur tract is one of them. The Madhupur tract is in between the present and old course of the river. The likely cause of the avulsion around the madhupur tract is a combination of tectonic movement and either a liquefaction flow that partially blocked the old channel, an increased flood discharge or a catastrophic flood in 1787. It is highly possible that the new course of river occupied an older Teesta River channel and the Teesta River itself finally became a right bank tributary of the Brahmaputra (RSP; 1996).

To describe the historic development of the river, three maps are presented in **Figure.1** The oldest one is known as the Rennel map, published in 1776, the second is Wilcox map surveyed in 1821-1834 and published in 1840. The recent map is prepared by ISPAN, 1991, from satellite images.

The Rennel's map clearly shows that in the second half of the 18th century the present Jamuna River was flowing at the Eastern side of the Madhupur tract and joined with the upper Meghna River before meeting with the bay of Bengal. The Wilcox map shows evidence that in the period between 1967 to 1830 a new channel evolved that was carrying the major part of the discharge. The old Brahmaputra river was still important for draining off the water. The Dhaleswary River, one of the left bank distributaries of the Jamuna River, was remarkably more prominent than in its present state. The planform of the river shown in 1830 was essentially a meandering river with very broad and low curvature meander loops. With an increased discharge and a heavy silt load, the stream gradually changed from meandering to a typical braided channel. During the past two decades (1973-1992) the river had widened while continuing to migrate towards the west.

Causes of river braiding

The reason why a river braides is not clearly known. Various opinion about the definition and causes of braiding are present in the literature. Leopold and Wolman (1957) defined a braided river as one which flows in two or more anastomosing channels around alluvial islands, whereas Lane (1957) stated, ' a braided stream is characterised by having a number of alluvial channels with bars and islands between meeting and dividing again, and presenting from the air the intertwining effect of a braid'. The general conditions required for the development of a braided planform described by knighton (1984) are as necessity of a large amount of sediment to provide an abundant bed load. Coarse fractions, which the stream is locally unable to transport, can provide the initial deposits of bars, which then divert the flow towards the river bank. Erodible banks are therefore another requirement as these are not only a readily available source of sediment but also allow the widening of the channel. There is good agreement regarding the requirement of easily erodible bank for braiding. Generally rivers with erosion-resistant banks meander rather than braid. The highly variable discharge is not a prerequisite for braiding. The fact that braided streams can be produced in flumes under steady flow conditions undermines the importance of the high variation of discharge. But the rapid variation of discharge often accelerates the bank erosion and thus contribute to braiding process (RSP, 1986).

In the literatures a number of methods of classification of channel patterns are avilable, which provided the threshold between the braiding and the other channel pattern. This can also provide a fair understanding of the causes of braiding. Leopold and Wolman (1957) provided a threshold value on the basis of bankful discharge and channel slope. In addition, recent studies on the channel classification based on stability analysis, indicate that braiding initiates from the instabilities in the river bed. There is a common threshold parameter in most of the channel pattern classification based on stability analysis, is the form ratio. For braiding the width depth ratio should be higher.

It appeared that braiding is likely for the river with highly erodible banks, higher slope discharge and width-depth Ratio. However, the width depth ratio is the function of sediment size, slope, sediment transport and characteristics of bank material.

Characteristics of braided river

The detailed processes giving rise to the formation of a braided river is still poorly understood. Many researchers studied the process of braiding but little agreement can be found, and the hydraulic parameters of the braided streams are extremely complex. Most of these works were based on small mountain rivers and flume studies and which are difficult to extrapolate to the large natural rivers, like Jamuna. Nevertheless, these works are significant for a general understanding of the processes in braided rivers. The large braided river is characterized by a wide channel, rapid movement of bed materials and continuos shifting of the channel course. Bed material shifting associated with the sorting of the grains is one of the requirements for the braided form. The channel shifting process is quite pronounced and rapid at the higher stages of flow and less during low flow. Chien (1961)pointed out that the amount of shifting i.e the lateral movement is controlled by the spacing of controlling points along the river. The controlling points are the bed rocks, and resistant strata on the banks. The continous shifting of the channel is attributed to the rapid movement of the bed material associated with the erosion and deposition of the river bed sediment.

According to the Leopold and Wolman (1964) ' there is a close relation between braiding and meandering : braided channels may exhibit curves that have a characteristic relation of radius to channel width, and the river has at least some reaches that would be called meandering. This can be explained by the division of flow of a bar resulting in the reduction of discharge in an individual channel and consequent reduction of the flow power, which causes the meandering.

Summarizing the above it appears that the bed of braided river is extremely unstable. The channel pattern classification based on stability of analysis also shows that braiding is initiated by instability of the river bed i.e. instability is inherent in the braiding process. This does not imply that the river itself is unstable. Rather it may be as close to equilibrium as are river showing meandering or other pattern (Leopold and Wolman 1964).

Bed form and roughness

The bed forms are the result of the channel bed modification to achieve the most efficient balance between sediment and water discharge i.e. the bed form is the action of a river bed to discharge water and sediment mixture of a given situp of a river (the valley slope, bed material, planform) with the variation of water and sediment input. The different types of bed modification occur depending on the flow regime. Accordingly the flow regime is classified as lower, transition, and higher flow regime. This classification is based on the form of bed configuration., mode of sediment transport, process of energy dissipation and phase relation between the bed and water surface. Generally, the bed roughness i.e the resistance of flow depends on the types and shapes of the bed form (RSP, 1996).

Van Rijn (1982) also provided the method of roughness estimation from the bed form size and shape dividing the effective roughness height of Nikuradse (k_s) into two parts. One is the grain related part (k_s') and the other is the form related part k_s'' which are as follows :

$$k_s=k_s'+k_s'' \qquad (1)$$

Then Chezy's roughness co-efficient (C) can be estimated as :

$$C=18\log (12h/k_s) \qquad (2)$$

where h= water depth, Van Rijn proposes for grain roughness:

$$k_s'=3D_{90} \quad \text{for } \theta<1 \text{ (lower regime flow)} \qquad (3)$$

$$k_s'= 3\theta D_{90} \quad \text{for } \theta>1 \text{ (upper regime flow)} \qquad (4)$$

where θ=Shield parameter, In lower flow regime, the bed form that contributes significantly to roughness is dune. Dunes are defined by Van Rijn as asymmetrical bed forms with a length of about 7 times the water depth or which roughness is proposed as,

$$k_s'' = 1.1\,\gamma_d\,H(1-e^{-25H/L}) \qquad (5)$$

Where, H= dune height, L= dune length and γ_d =form factor, generally 0.7 for field condition. The dune height estimated by Van Rijn (1982), based on flume and field data are as follows :

$$H = 0.11\, h^{0.7}\, D_{50}^{0.3}\, (1-e^{-0.5T})\,(25-T) \qquad (6)$$

According to Simons and Richardson (1966) and Van Rijn (1984), as the shear stress (τ_b) increases, the flow regime steps up from lower to high flow regime and subsequently the bed form changes from ripples to sand wave and anti-dune. The bed form dimensions depend on the flow depth, grain size and also bed shear-stress (Equation 6) and finally the roughness is the function of the size, shape of the bed form.

Due to the higher variability of flow condition in a natural river, the bed forms generally differ from the classification shown above. Coleman is the first who provided the data on the bed forms in the Jamuna River which are distinguished as follows (RSP ; 1996) :

Ripples	:	Typical height is 0.2 to 0.5m
Mega ripples	:	Typical height is 1 m and celerity is 120 m/day
Dunes	:	Typical height is 5 m and celerity is 60 m/day
Sand Waves	:	Typical height is 10 m and celerity is 200 m/day

Erosion and deposition processes in the river bed

Different studies on the Jamuna River shows that the channel processes in the Jamuna River are quite rapid and generally associated with erosion and deposition with river bed. In such a quickly responding river with comparatively low bed load transport, whose processes of sediment transport i.e the bed, near-bed or suspended load transport activate the changes of river bed left a question to model morophological processes of the river.

Channel shifting processes

The channel shifting processes in the Jamuna River are quite rapid. The abandonment of large channels and the developing of new channels in a few years only are common features in the Jamuna River. Klaassen and Masselink (1992) found that three types of processes are related to channel shifting processes, viz. bar-induced shifting, development of cutoff and outer bend channel in bends. Among these, the development of the cutoff in the channel shifting process happens quite often in the Jamuna River.

Klaassen and Van Zanten analytically studied the cutoff process in the meandering channels. Their study was based on the ratio of the sediment supply from the offtake into the cutoff channel and the sediment transport capacity of the channel (RSP; 1996). They found that the most important parameter is the cutoff ratio (λ), which they defined as the ratio of the length of the channel and the length of the cutoff channel. Later study by Klaassen and Masselink (1992) based on the analysis of satellite images in the

Jamuna River, found that for cutoff the ratio λ varies from 1 to 1.7 and the average value of the cutoff ratio as they suggested is 1.25. The cutoff ratio in the Jamuna River is very low in comparison to the meandering river, where the cutoff ratio of 5 to 30 is common, suggesting that cutoff occurs very quickly in the Jamuna River.

Bank erosion and its process

According to Coleman (1969) two types of bank failure generally occur in the Jamuna River : liquefaction and flowage of material, and shearing away of bank materials. The former type of bank failure can occur below the low water level or in the zone of low and high water level. Generally they occur during the receding of flood discharge. Receeding rates of water level directly influence the rate of failure. The most common process of bank failure in the Jamuna River is due to shearing, caused by flow attacking the bank or over steepening the bank by a thalweg approaching the bank. However, the bank erosion and accretion rate provided by Coleman (1969) at an interval of 5 to 8 km apart for two different periods, 1944-1952 and 1952-1963, based on the analysis of three maps, showed that the bank erosion rate was erratic and varied from 0 to 800 m/year. Taking the bank accretion as zero, the average bank erosion rates of the Jamuna River based on Coleman are presented in Table 1 (RSP ; 1996).

Table 1: The average bank erosion rate of the Jamuna River (RSP,1996)

Bank	Bank erosion rate (m/year)	
	Period	
	1952-1963	1944-1952
Left	43	118
Right	67	177

The average bank erosion rate at the right bank is higher for the two periods, consistent with the westward migration of the Jamuna River. The period 1944-1952 showed an average bank erosion rate of more than two times the later period, illustrating the variability with time of bank erosion rate. By analyzing satellite images Thorne et al. estimated the bank erosion rate of the right bank of the Jamuna River for the period of 1973-1992. They estimated bank erosion at an interval of 500 m apart and averaged the erosion rate for 10 km. The average bank erosion rate for the period 1973-1992varies from 0 to 160 m/year and the average erosion rate is about 80 m/year. They found that for a shorter time scale the bank erosion rate increases, and the average bank erosion rate is higher for a higher number of flood discharges. They also found that catastrophic events of bank erosion (>350 m/year) generally occur for short duration : 2 to 4 years and this erosion took place at the outer bank of the curved channels.

Klaassen and Masselink (1992), studied the bank erosion rate of the curved channels in the Jamuna River by analyzing the satellite images from 1976 to 1987. As the Jamuna River is a braided river, the various braided channels having different discharges, width and radius of curvature are usually active in eroding the bank at different places. They found that in the Jamuna River the bank erosion is generally associated with the rotation and extension of bend rather than translation.

A good understanding of bank erosion processes and bank erosion rates is important for the prediction of future planform changes and, hence, implicitly for the protection of levees, of flood protection structures or of other hydraulic structures in the river. Bank erosion is one of the most complicated processes in river morphology and even more complexity arises as in a braided river like the Jamuna, where the location of bank erosion may vary quickly in time and space.

River bank erosion is a complex process in which many factors are involved. 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 material properties. The flow exerts shear stresses that can remove particles from the bank either via peeling off or via mass movement. The near bank flow pattern is determined by the flow and channel geometry. Bank material properties determine the cohesivness of the bank, an important parameter for the type of bank erosion, and is also important for how quick erosion products are transported by the river and thus determine the time needed for the typical cycle toe-erosion-failure-transport important for mass failures. Vegetation does not play an important role in bank erosion processes along the Jamuna River. Groundwater flow may have an important effect on the bank erosion, especially during the recession of flood. The flow in a river bend attacks the toe of the river bank, removing the sediment from the toe resulting in an over-steepening of the river bank and causing the bank failure by slumping.

Conclusion

The Jamuna river is an extremely dynamic river, changes in the upstream reaches are able to cause enormous changes in the downstream river reach. The morphology of the Jamuna river is greatly influenced by flood and low flow season. During all stages morphological changes taking place, owing to the bed material consisting of fine sand, morphological changes during the flood season are rapid. In this wide and untrained sand-bed river, deposition and erosion of sediment takes place on a major scale.

The changes in flow direction, channel topography and planform, the occurrence of new channels and abandonment of old channels, changes of other morphological features like change of bank erosion rate, discharge dependent scour holes etc. are quite pronounced. The inherent process, which result in the observed behavior of the river, are the continuos redistribution process of the sediment within the channel as a consequence of the constantly changing discharge conditions and other changes in boundary conditions.

The braided rivers are unstable in nature and braiding is initiated from the instability of river bed. Generally erodible banks are prerequisite for braiding including the higher discharge, slope and form ratio.

References

Chien, N. (1961) **The braided stream of the lower Yellow river**, Scientia Sinica, V. X. no.6.

Coleman, J. M. (1969) **Brahmaputra river; Channel Processes and Sedimentation**, Sedimentary Geology, Vil.3 Nos. 2-3.

Khan, Z.A. (1995) **Statistical Characteristics of discharge and sediment transport in the Ganges and Jamuna Rivers**, M.Engg. Thesis, Dept. of Water Engg. BUET, Dhaka, Bangladesh.

Klaassen, G.J. and Masselink, G. (1992**) Planform changes of a braided river with fine sand as bed and bank material**, Proc. 5th International Symposium on River sedimentationKarlsruhe, FR, Germany, pp 459-971.

Klaassen G.J. and Vermeer, K. (1988**) Channel characteristics of braided Jamuna rivers**, Proceedings of International conference on River Regime, Wallingford, U.K., pp. 171-189.

Knighthon, D (1984) **Fluvial forms and Process**, Edward Arnold, London, pp. 90-147

Leopold, L.B., Wolman, H.G. and Miller, G.P. (1964) **Fluvial Processes in Geomorpholog**, Freeman, San Francisco, pp.522.

Lane, E.W. (1957) Study of the shape of channel formed by natural streams flowing in erodible materials, Missouri river division sediment series no.9, U.S. Army Engineer Division, Missouri river, Corps of Engineer, Omha, Nebrasca.

Leopold, L.B. and Wolman, M.G. (1957) **River channel patterns, braided, meandering and straight**, U.S. Geol. Survey, profess. Papers, 282-B.

River Survey Project (1996) **Morphological Processes in the Jamuna River**, Special report no.24, Dhaka, Bangladesh.

River Survey Project (RSP) (1995a) **Sediment rating curves and balances**, Special report 18, Dhaka, Bangladesh.

Thorne, C.R, Russel, A.P.G and Alam, M.K., (1993) **Planform Patterns and Channel Evaluation of the Brahmaputra river**, *Bangladesh in Braided Rivers"* J.L, Beast and C.S, Bristow (eds), Geological Society of London, special Publication no.75, ISPN 0-903317-93-1, pp.257-276.

Van Rijn, L.C. (1982**) Prediction of Bed forms, Alluvial Roughness and Sediment Transport**, *Delft Hydraulics,* S 487 III, Delft, The Netherlands

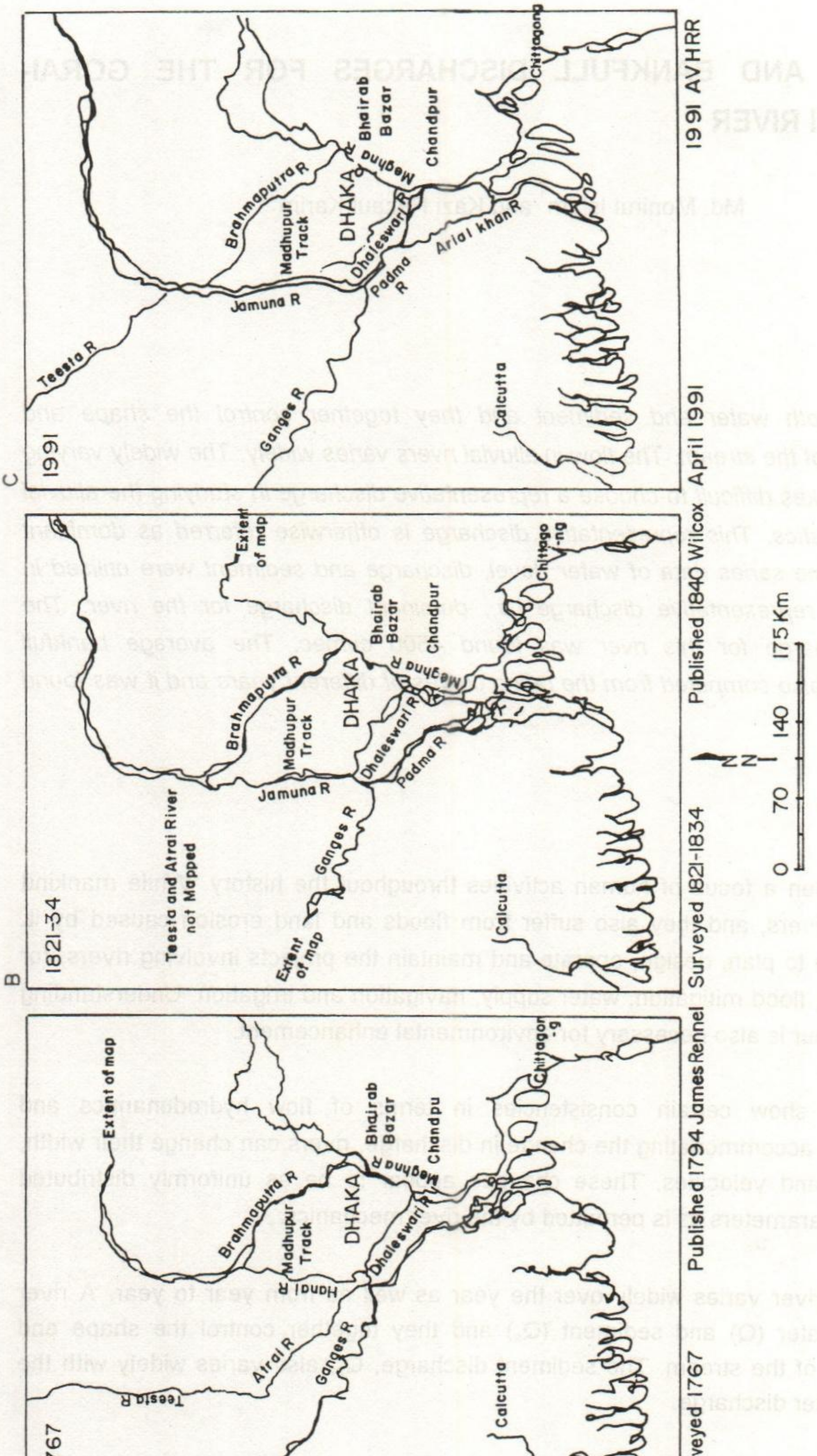


Figure 1 : Historic maps of the river system of Bangladesh (RSP, 1996)

DOMINANT AND BANKFULL DISCHARGES FOR THE GORAI-MADHUMATI RIVER

Md. Monirul Islam[1] and Kazi Rezaul Karim[2]

Abstract

Rivers carry both water and sediment and they together control the shape and characteristics of the stream. The flow in alluvial rivers varies widely. The widely varying flow pattern makes difficult to choose a representative discharge in studying the alluvial river characteristics. This representative discharge is otherwise referred as dominant discharge. A time series data of water level, discharge and sediment were utilised in estimating the representative discharge i.e., dominant discharge for the river. The dominant discharge for this river was found 4500 cumec. The average bankfull discharge was also computed from the rating curves of different years and it was found 4813 cumec.

Introducaiton

Rivers have been a focus of human activities throughout the history. While mankind benefits from rivers, and they also suffer from floods and land erosion caused by it. Engineers have to plan,. dèsign, operate and maintain the projects involving rivers, for their regulation, flood mitigation, water supply, navigation and irrigation. Understanding of river behaviour is also necessary for environmental enhancement.

Alluvial rivers show certain consistencies in terms of flow hydrodunamics and morphology. In accommodating the change in discharge, rivers can change their width, depth, slopes and velocities. These changes appear to be as uniformly distributed among these parameters as is permitted by the river mechanics.

The flow in a river varies widely over the year as well as from year to year. A river carries both water (Q) and sediment (Q_s) and they together control the shape and characteristics of the stream. The sediment discharge, Q_s, also varies widely with the variation of water discharge.

---

[1]Senior Scientific Officer, RRI, Faridpur; [2]Senior Scientific Officer (In Charge), RRI, Faridpur

The ratio of maximum to minimum discharge may be as high as 1000 or more (Islam,1996). The flow in the medium river varies widely. The ratio of maximum to minimum discharge for both water and sediment may be as high as possible because of very low flow in the dry season. But to maintain an equilibrium condition for alluvial channels, it is widely recognised that this ratio for both water-sediment discharge should be as low as possible.

Problems of river morphology are very complex. For this reason, simplification of the river system is a must. But, the simplified river regime can never replace the real regime as far as the reproduction of the morphological characteristics of a river reach are concerned. The widely varying flow pattern makes it difficult to choose a representative discharge in studying the characteristics of the alluvial river.

Different methods have been proposed by diferent investigators for the choice of representative discharge. The dominant discharge concept has widely been used for an efforts to define a river regime into one single discharge.

Bankfull discharge has been often assumed to be a significant or critical channel forming flow which is important in determining the size and shape of the river channel and it has therefore sometimes been regarded as equivalent to dominant discharge (Goudie, 1985).

The Gorai-Madhumati river

The Gorai - Madhumati river is one of the main distributaries of the Ganges flowing through the deltaic plains. The Gorai takes off from the right bank of the Ganges near Talbaria, Kushtia in the name Gorai and it flows south-eastwards and falls into the Bay of Bengal through the Haringhata estuary. The total length of the river, upto the down of Patgati, is about 258.5 km (161.5 miles). The river has different names at different parts of its course. The upstream part of kamarkhali ghat is known as the Gorai, down stream of kamarkhali ghat is named by Madhumati and further down stream of Patgati, it takes the name Baleswar. This is a moderately old river. From the history, it is found that the river is more than 230 years old. In tracing the changes in the rivers in Jessore and Faridpur districts, Major James Rennell's (a surveyor of British Govt.) found the Kushtia creek (Gorai) in June 1764. The river was 130 to 200 yards wide and 15 to 60 ft. deep near off-take. The depth had been reduced very sharply inside the country. Rennell (1767) and later on Williams (1919) mentioned the name of the Nabaganga and Barassia without mentioning the Madhumati. The lower reaches of the Barassia appeared from the Rennell's map (1767) to have followed much the same course as the the Madhumati does now (Williams, 1919).

Westland's (1870) after Williams (1919) stated that the Madhumati probably was the out come of the severe floods which swept over the district between 1795 and 1801. On the other hand Fergusson (1863) after Williams (1919) stated that the Madhumati formed some time between 1818-20 and 1830-33. The statement in Westland's history may incorrect as Fergusson possessed a close knowledge of these river at the time the changes took place (Williams, 1919). Before 1911-12 probably the rivers Gorai and Madhumati were the different rivers. Location of the Madhumati was in the east and that of Gorai was on the West (Williams, 1919).

Present flow condition

The flow of the Gorai-Madhumati river largely depends upon the hydrodynamic and morphological conditions of the Ganges near the Gorai off-take. During the last three decades, diversion of flow of the Ganges to the Gorai has been highly variable due to moving sandbars which seriously obstruct the flow into Gorai periodically. The average annual discharge at Gorai Rly bridge and at Kamarkhali are 1409 and 1255 cumec respectively. The average minimum flows at the Gorai Railway bridge and at Kamarkhali before 1975 were 87.32 and 108.50 cumec respectively and those after 1975 are 28.83 and 25 cumec respectively. The minimum flows are 0.057 cumec and 0.764 cumec at Gorai Railway bridge and at Kamarkhali transit respectively. The average discharge in winter at Gorai-Railway bridge was 475.42 cumec and in rainy season 6035.35 cumec in the year 1969 and the maximum discharge was 7568 cumec. In 1988, the year of devastating flood in Bangladesh, the highest discharge was 8490 cumec at Gorai Rly bridge site. On the basis of the data available from 1946 to 1990, BWDB, the recorded highest discharge at Gorai Railway bridge is 8490 cumec in 1988 and at present even no flow occurs during the lean period. Actually in winter no water of the Ganges falls into the Gorai and hence the upstream parts remain almost dry. But in the rainy season huge amouunt of water from Ganges river pass through the Gorai river (Islam,1996).

The minimum discharge at Gorai-Railway bridge in 1988 was only 0.286 cumec on the contrary the maximum discharge in the same year was app. 8490 cumec, i.e., the ratio of maximum discharge to minimum discharge was approximately 30,000, which is too high. That is, discharge variation throughout the year is as high as possible.

Dominant discharge

Originally the dominant discharge concept was introduced in combinations with the regime theory in order to extend the regime theory to those rivers in which discharge change and vary throughout the year (Inglis, 1949). At this discharge, equilibrium is most closely approached and the tendency to change is the least.

Inglis (1947), defined the dominant discharge as that hypothetical steady discharge which would produce the same result as the actual varying discharge. Inglis suggested that for most of the Indian rivers the dominant discharge was nearly the same as bankfull discharge and further recommended that the dominant discharge may be taken to be half to two thirds of the maximum discharge.

Schafernok (1950) after Hossain (1992) proposed based on the reasoning that the dominant discharge should be the discharge at which most of the formative works is done and should "therefore" correspond to that stage at which the bulk of the bed load is carried. But it is difficult to decide at which stage the bulk of the bed load is carried.

Leopold and Maddock (1953) preferred to use a discharge of particular frequency of occurrences for comparing the hydraulic geometry of streams. According to them the mean annual discharge which was found to be equalled or exceeded about 25 percent of time was most suitable for studying the variation of hydraulic geometry along the river.

Blench (1957) designated the dominant discharge as that discharge which is equalled or exceeded 50 percent of time. This definition is valid for American rivers.

Nixon (1959) preferred to use bankfull discharge for studying the alluvial channel geometry in England and Wales. He found that the average frequency of this discharge is about 0.6 percent. As this frequency is a function of climatic condition, drainage characteristics and river order which are highly variable, departures from the above values are likely in other rivers.

Ackers and Charlton (1970), defined dominant discharge as the steady flow that would produce the same meander wave length as the observed range of flows within which that steady flow lies. This definition relates the dominant discharge with a new parameter, the meander length.

Wolman and Gerson (1978 after Halcrow and Partners 1991), extended the Wolman's original contribution on dominant descharge in arguing that the effectiveness of flow event reflects the morphological changes it causes through erosion and deposition, as well as the associated sediment transport. These are, however, complementary definitions in that it should be expected from basic principles that the flow doing most of the work as the channel would be responsible for forming a scaling the salient parameters of its geometry, size and sedimentary features.

During the study of the Brahmaputra for Jamuna Multipurpose Bridge the dominant discharge was defined as "that steady discharge which had it operated continuously for the period of record, would have transported the same amount of sediment as the range of flows which actually occurred".

The analysis would be done according to the approach suggested by Schafernok (1950) and the result may be compared with the Blench (1957) flow duration concept.

Bankfull discharge and bankfull stage

The river discharge which exactly fills the river channel to the bankfull level without spilling on to the flood plain. Bankfull discharge has been often assumed to be a significant or critical channel forming flow which is important in determining the size and shape of the river channel and it has therefore sometimes been regarded as equivalent to dominant discharge (Goudie, 1985). But in terms of sediment transport specifically of bed load transport studies from different river have demonstrated that the dominant discharge for bed load transport is not the same as the bankfull discharge. Therefore, the discharge at the bankfull stage should not be thought of as the formative discharge (Petts, 1983). Always the bankfull discharge approximated the discharge slightly higher than the actual dominant discharge estimated from the bed load transport considerations. Nevertheless, it provides a working approximation of the dominant formative discharges. The frequency of occurrence of bankfull discharge varies considerably and has been equalled as ranging from 4 months to 5 years in Scotland, 1.3 to 14 years in lowland England and 1 to 30 years in 28 basins in the Western USA (Nixon, 1959).

On hydrological grounds there is a significance of the channel bankfull stage. Flow resistance decreases as water depth within the channel increases; reaches a minimum at bankfull stage; and increases again as the water flows out of the channel and on to the adjacent flood plain. Various possible definitions of the bankfull stage (at least 16 different methods) have been used (Williams, 1978a) referring to : the heights of the valley flat, the active flood plain, the benches within the channel, the highest channel bars, the lower limit of perennial vegetation, the upper limit of the sand sized particles in the boundary sediment, the elevation at which the width-depth ratio becomes a maximum, where there is a first maximum of the bench index (Riley, 1972 after

Goudie, 1985), or the relation of cross-sectional area to top width changes. Williams (1978a) concludes that the bankfull level to the active flood plain level is the most useful of the fluvial geomorphologists, whereas the banks of the valley flat are the most important to engineers.

Sediment transport

The most important of a river is that it prevails a movement of water, and sediment from the land to the oceans consequently the structure and form of the land and its margins are altered through erosion and deposition. Then character of the river changes. Sedimentation includes the processes of erosion, entrainment, transportation, deposition, and the compaction of sediment in the stream channels. Quantification and investigation of the properties of sediment transported through alluvial channels are an important task, because they play a vital role on the stability and behaviour of an alluvial stream. The representative discharge or dominant discharge of a natural stream depends completely upon the characteristics of the sediment transported through the channel. The theories and empirical relationships for the prediction of sediment movement so far developed are quite large Islam (1996).

In estimating the dominant discharge sediment transport data is of utmost importance. Two important sediment discharge formulas, e.g., Engelund and Hansen (1967) and Hossain (1987), are best suited (walliuzzaman, 1986) and easy to handle in computing the sediment discharge of Gorai-Madhumati river were used.

Engelund and Hansen (1967) formula

Engelund-Hansen (1967) developed a sediment discharge formula based on the shear stress approach.

The equation can be written as :

$$\gamma_s = 0.05\times\gamma_s\times V^2(d_{50}/(g(\gamma_s/\gamma-1))^{0.5}\times(\tau_0/(\gamma_s-\gamma)\times d_{50}))^{3/2} \quad \text{...(1)}$$

Where, g_s = The sediment transport per unit time per unit width,
v = the average flow velocity,
d_{50} = median particle diameter,
τ_0 = bed shear stress,
γ_s = specific weight of the sediment particles,
γ = specific wt. of the water.
g = acceleration due to gravity.

All the variables were in the F.P.S. system.

Hossain (1987) formula (after Dey, 1995)

In 1987 Hossain suggested a sediment transport formula based on the concept of dimensional analysis and similitude argument.

The equation can be written as :

$$C_t = A(X^a Y^b Z^c) \quad \text{(2)}$$

Where, C_t = Sediment concentration in ppm,

$A = 0.845 \times 10^5$ for $Q < 1.0$ m^3/s
$= 6.946 \times 10^5$ for $Q > 1.0$ m^3/s and $B/H < 100$
$= 6.946 \times 10^6$ for $Q > 1.0$ m^3/s and $100 < B/H < 500$

$$X = vs/(gh)^{1/2} \quad \text{(3)}$$

$$Y = w_r/w \quad \text{(4)}$$

$$Z = Q/Q_c \quad \text{(5)}$$

and

a = 0.745
b = 0.633
c = 0.500

w_r = Settling velocity for a representative sediment size for which d_{50} = 0.15 mm at ambient temperature.
w = Settling velocity of the sediment particles.
Q_c = Assessed discharge from equation.

$$Q_C = [\,(2.15 + K\,B/H)H\,(gs)^{1/5}\,]^{5/2} \quad \text{(6)}$$

where, Q = discharge
K = 0.205

But this formula was modified when applied against large river data of Bangladesh, value of K were found as follows.

K = 0.055 when $Q < 15000$ m^3/s
K = 0.170 when $Q > 15000$ m^3/s
B = average width of channel
H = average depth of channel
s = average water surface slope
g = acceleration due to gravity

Dominant discharge for The Gorai-Madhumati river

Estimating the dominant discharge according to the Schafernok (1950) after Hossain (1992) concept sediment rating curves and flow frequency distribution curves (as in Figure 1) were constructed first. Then with the help of these curves sediment transport Q_s vs discharge Q curves (as in Figures 2 and 3) were constructed for Gorai-Railway bridge site. A flow duration curve (Figure 4) was also constructed to observe percentage of time the dominant discharge equalled or exceeded. The peak of the curves (as in Figures 2 and 3) defines the flow doing most of the work on the channel through the transportation of sediment i.e. the dominant discharge.

From Figures 2 and 3 it is appeared that the dominant discharge according to this concept to be 4500 m^3/s. The analysis, using different discharge classes, different sediment rating curves etc. give approximately the same result. And this is quite a good result which is not sensitive to the precise nature of the sediment rating curves used to derive it.

Table 1 reveals that the average discharge of different years (1965-88) corresponding to the bank-full stage (+12.19m PWD) to be about 4813 m^3/s. Which is slightly higher (only 6.9 %) than the discharge found earlier(4500 m^3/s)). The highest yearly reported discharges (peak discharges) at Gorai-Railway bridge site for the period of 1965-88 as per BWDB records were also noted. There is a concept that the dominant discharge varies between the ranges of half to two-third of the peak discharge (Islam, 1996).

It appears from Table 1 that the average value of half of the peak discharges was 3214 m^3/s and an average value of two-third of peak discharges was found to be 4285 m^3/s. An average of them to be 3750 cumec which is about 58% of the average peak discharge and some what less (by 16.67 %) than that of discharge found earlier by Schafernok concept.

From flow duration curve (developed utilising the data for the period of 1967 to 1988 for Gorai-Rail Way bridge site) it appears that the discharge 4500 m^3/s equalled or exceeded about 11.25 % of the time and the discharge 3750 cumec equalled or exceeded approximately 15% of the time. It is appeared that the frequency of occurrence of the representative discharge (D.D.) were appreciably low in the Gorai-Madhumati river which is doing most of the work on the channel, i.e., development of meander, adjustment of meander length, meander belt etc. It is therefore, concluded that the dominant discharge of the Gorai-Madhumati river is about 4500 m^3/s, which is equalled or exceeded approximately 11.25 % of the time. But dominant discharge found for the Gorai was approximately 4250 m^3/s by the almost same procedure for South West area water resources management project study (Halcrow & Partners, 1993).

Conclusion

Based on the present study, it may be concluded that the dominant discharge of the Gorai-Madhumati river was 4500 cumec and the bankfull discharge was 4813 cumec. The analysis using different discharge classes, different sediment rating curves etc. give approximately the same result. And this quite good result which is insensitive to the precise nature of the sediment rating curves used to derive it. On the other hand frequency of occurrance of the dominant discharge was appreciably low in the Gorai-Madhumati river which is doing most of the work on the channel.

Acknowledgements

The resulting data presented herein, unless otherwise noted, were obtained from research conducted under the Department of Water Resources Engineering, BUET for the Master of Science in Water Resources Engineering thesis. The authors would like to express their deepest gratitude to Prof. Dr. Ainun Nishat and Prof. Dr. Md. Monowar Hossain.

References

Ackers, P. & Charlton, F.G. (1970) **Meander geom. arising from varying flows**, Journal of Hydrology. Vol. 11.

Blench, T. (1957) **Regime behaviour of canals and rivers**, Butterworths scientific , pub London, England.

Dey, K.C. (1995) **Schematise sediment transport predictor for study of alluvia rivers : An application to the Jamuna**, M.Sc. Engineering thesis, Dept. of WRE BUET, Dhaka.

Engelund, F., Hansen, E.A. (1967) **A monograph of sed. trans. in alluvial streams**, Teknisk Vorlag.

Goudie, A., Atkinson, B.W., Gregory, K.J. & Simmons, I.G. (1985) **The encyclopaedic dictionary of physical geography**, Basil Blackwell Ltd., Oxford OX4 1JF, England.

Halcrow and partners Ltd., Sir william, et al, (1991) **River training studies of the Brahmaputra river**, Annex-4, BWDB, Dec.

Hossain, M.M. (1992) **Dominant discharge of the Ganges and Jamuna**, Journal of the Institution of Engineers, vol. 20, #3, pp.7-12.

Inglis, C.C. (1949) **The behaviour and control of rivers and canals**, Pt. 1, CWPINRS, Res. pub. No. 13, Poona.

Inglis, C.C. (1947) **Meanders and their bearing on river training**, Proc. of the Inst. of civil Engrs. London, Maritime & Water Engg. div.

Islam, M. M. (1996) **A study on the morphological characteristics of the Gorai-Madhumati river**, unpublished M.Sc. Engg. thesis, Dept. of WRE, BUET.

Leopold, L.B. and Maddock, T.Jr. (1953) **The hydraulic geometry of stream channels and some physiographic implications**, U.S. Geological Survey professional paper 252,p 57.

Nixon, M. (1959) **A study of the bankfull discharge of rivers in England and Wales**, Proc. of the Inst. of civil Engrs., vol 12, London, England, Feb, pp-157-174.

Petts, G.E. (1983) **Rivers sources and methods in geography**, Butterworths.

RPT/NEDECO/BCL (1990) **Jamuna Bridge Project**, River training works, Design report, vol. 2, Report on model investigation, JMBA.

Walliuzzaman, M. (1986) **Study of sed. trans. in the river Gorai-Madhumati**, M.Sc. Engg. thesis, Deptt. of WRE, BUET.

Williams, C.A. (1919) **History of the rivers in the Gangetic delta 1750-1918**, NEDECO, Reprint, Dhaka 1966.

Williams, G.P. (1978a) **Bakfull discharge of rivers**, water resources research 14, pp. 1141-54.

Table 1 : Discharges and their components for the Gorai-Madhumati river at Gorai Rly bridge station between 1965 and 1988.

Year	Q_{msx} m^3/s	$Q_{75\%}$ of Q_{max} m^3/s	$Q_{50\%}$ of Q_{max} m^3/s	$Q_{60\%}$ of Q_{max} m^3/s	Bankfull Discharge From Rating Curve (m^3/s)
1965	4770	3577	2385	2862	4100
1966	5850	4387	2925	3510	4590
1967	6880	5160	3440	4128	4800
1968	6280	4710	3140	3768	4900
1969	7560	5670	3780	4536	5050
1970	5070	3802	2535	3012	4530
1971					
1972	5040	3780	2520	3024	5040
1973	7190	5392	3595	4314	5900
1974	8460	6345	4230	5076	6000
1975	5970	4477	2985	3582	5090
1976	6620	4965	3310	3972	5020
1977	5890	4417	2945	3534	4950
1978	6960	5220	3480	4176	4650
1979	4250	3187	2125	2550	4250
1980	6570	4927	3285	3942	4500
1981	6630	4972	3315	3978	5390
1982	6810	5907	3405	4086	4900
1983	7480	5610	3740	4488	4500
1984	6050	4537	3025	3630	4100
1985	6290	4717	3145	3774	4950
1986	5240	3930	2620	3144	4450
1987	7500	5626	3750	4500	4650
1988	8490	6367	4245	5094	4400
Avg.	6428	4821	3214	3857	4813

**Bankfull level considered was 40 ft or 12.19 m (PWD)

[Source: Islam,1996]

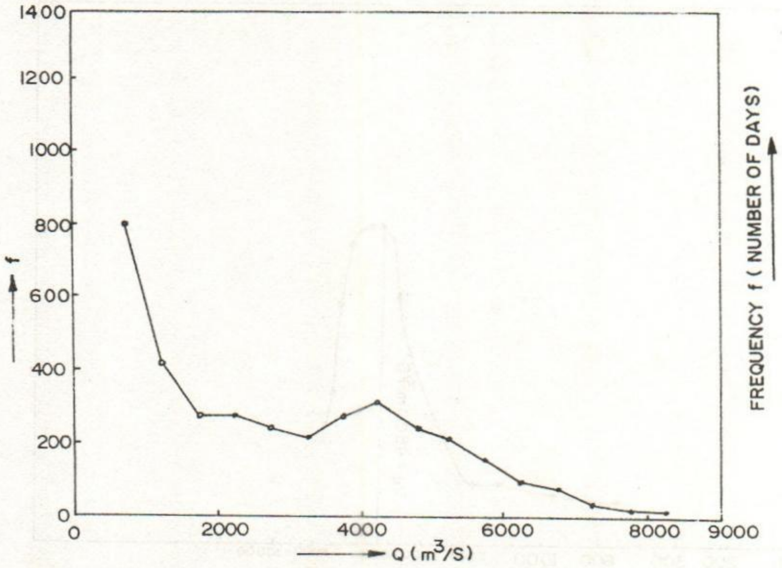


FIG. 1 : FREQUENCY DISTRIBUTION FOR DISCHARGE AT GORAI RLY BRIDGE FOR GORAI-MADH. RIVER
DATA RANGE 0-500, 500-1000, 1000-1500 ETC. (1965-89)
(SOURCE : ISLAM, 1996)

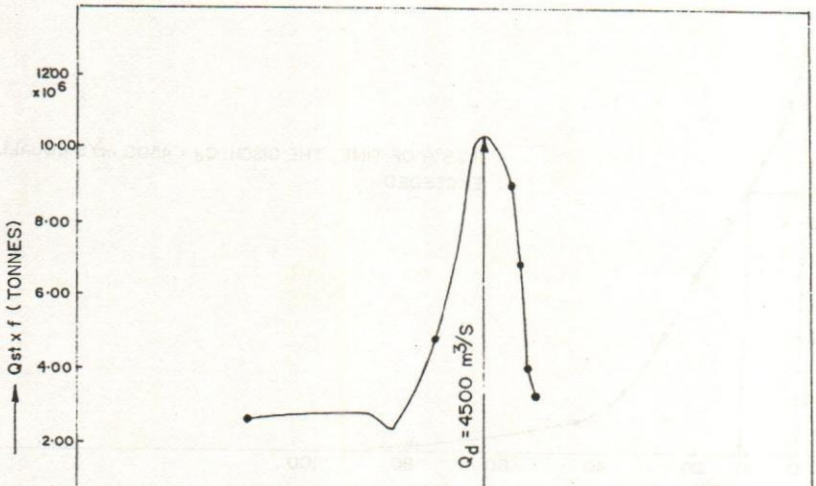


FIG. 2 : TOTAL SEDIMENT TRANSPORT (OBSERVED) Vs DISCHARGE CURVE AT GORAI-RLY. BRIDGE FOR GORAI-MADHUMATI RIVER WITH DATA RANGE AS 400-600, 800-1000 ETC. (SOURCE : ISLAM, 1996)

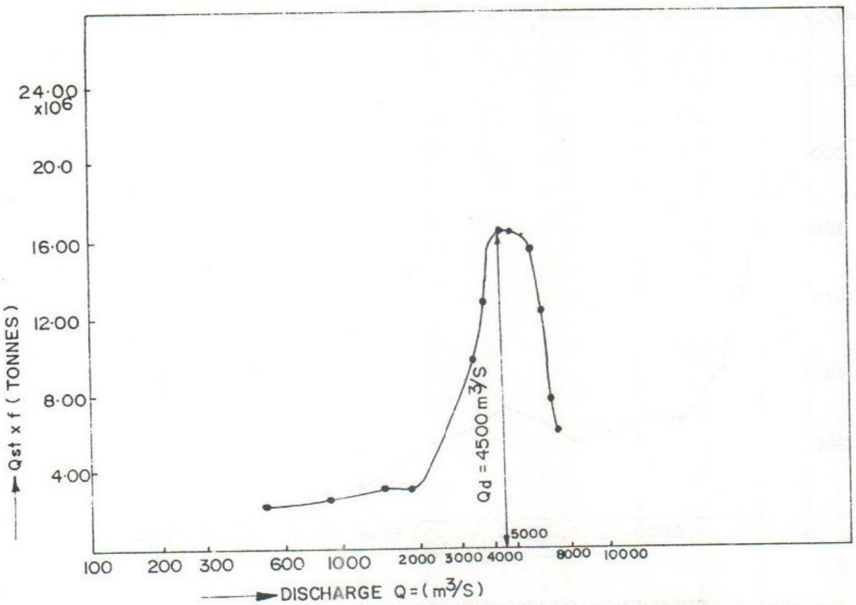


FIG. 3 : TOTAL SEDIMENT TRANSPORT (PREDICTED BY ENGELUND-HANSEN FORMULA) Vs DISCHARGE CURVE AT GORAI RLY. BRIDGE FOR GORAI MADH. RIVER WITH d50 = 0·130 mm AND DATA RANGE AS 400-600, 800-1000 ETC. (SOURCE : ISLAM , 1996)

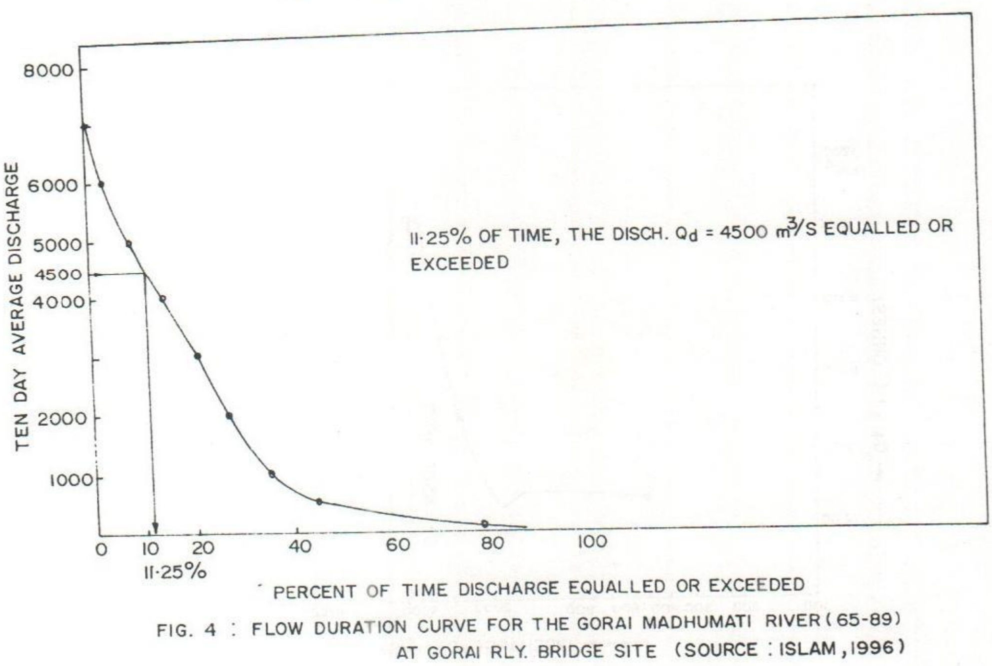


FIG. 4 : FLOW DURATION CURVE FOR THE GORAI MADHUMATI RIVER (65-89) AT GORAI RLY. BRIDGE SITE (SOURCE : ISLAM, 1996)

SEDIMENT CHARACTERISTICS OF THE GANGES, THE JAMUNA AND THE PADMA

Md. Nurul Haque[1], Md. Rafiqul Alam[2] and Md. Hanif Mazumder[3]

Abstract

Knowledge on source of sediment supply and sediment transport, characteristics of sediment and river morphology is essential prerequisite for planning and design of any river training works. This paper discusses sediment characteristics of the main rivers; the Jamuna, the Ganges and the Padma which obtained from laboratory study of the sediment samples collected by Bangladesh Water Development Board. This paper also deals with graphical representation of sediment transport rate with water discharge and some sediment characteristic parameters obtained from the study which is essential for mathematical model as well as for physical model set up. The key findings of this paper is that the bed material size D_{50} of the Jamuna is higher than that of the Ganges and the D_{50} of the Ganges is higher than that of the Padma. The geometric standard deviation increases as the silt content increases. So the higher value of geometric standard deviation indicates the higher percentage of silt content in the sample.

Introduction

Bangladesh is a land of rivers. The Brahmaputra-Jamuna, the Ganges-Padma and the Meghna are the three main rivers of Bangladesh. These main rivers and their distributeries and tributaries play an important role in the economic development of Bangladesh. Most of the important cities, towns and so many hydraulic structures are situated on the banks of main rivers of the country. In every year bank protection works are carried out to protect the river banks from erosion. Again many important projects are also taken up for irrigation facilities on both sides of the main rivers. A planner and a design engineer must have sound knowledge about sediment characteristics such as sediment concentrations, sediment transport rate, grain size of bed materials, bed load material and suspended material at the time of planning and design of hydraulic structures like, barrage, embankment, irrigation canal, drainage channel, reservoir, regulator, closure, river bank protection works or any other river engineering works. The characteristics of bed and the bank material is one of the most important parameters of an alluvial river. Many properties of the rivers are determined by the size and other characteristics of this bed material. The sediment transport load is inversely related to size of the material, the coarser the bed material, the smaller the sediment transport. The deposition of sediment also depend on its size. The bar deposits, composition of the banks and flood plains, the channel geometry and planform of the rivers are influenced by the bed material characteristics. Hence, a good knowledge of the bed material of the main rivers is very essential for a better understanding of the sedimentological and morphological process in the main rivers of Bangladesh.

---

[1]Senior Scientific Officer, RRI, Faridpur; [2]Principal Scientific Officer (In Charge), RRI, Faridpur
[3]Director (In Charge), Geotechnical Research, RRI, Faridpur

Jansen et.al. (1979) stated that according to the source sediment can be classified into two groups namely bed material load and wash load.

But according to mode of transportation, the sediment can be classified into two categories namely Bed load and Suspended load. Suspended load consists of sediment particles held in suspension by balancing their gravitational force with the upward forces due to turbulence of the fluid. and the wash load is defined as the transport of material finer than the bed material. The wash load depends on the amount of fine sediment which erodes from the upstream catchment area and it is independent of the transportation capacity of the river. The particles of wash load has diameter D_{50} less than 50 micron. Wash load is almost uniformly distributed throughout the river. Bed load is characterized by the movement of bed material by rolling, sliding and small jumps. Bagnold defines (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.

In this paper attempts have been made to determine the bed material and suspended material characteristics, sediment transport rate and some sediment characteristic parameters of the Ganges, the Brahmaputra-Jamuna and the Padma by laboratory studies on the sediment samples collected from these rivers. Some comments on findings obtained from studies have also been presented in this paper.

A short description of basin characteristics of the Jamuna, the Ganges and the Padma

The Jamuna

The Jamuna takes its rise on the northern slopes of the Himalayas as the Tsang-po-river. It runs through Assam of India as Brahmaputra. Finally near the Bangladesh border, the river runs south until it meets the Ganges at Aricha (Fig 1). The total length of the river is 2700 km and the length from the Indian border to its confluence with the Ganges is about 240 km. Its drainage area is about 560000 km^2. In Bangladesh the average W.L. slope of the Jamuna river is about 7.6 cm per km for first 130 km and 6.5 cm per km further down-stream. The river bed consists of fine sand with D_{90}=0.30 mm (RSP, Annex-3, 1996).

The Ganges

The Ganges rises west of the Nanda Devi range in Himachal Pradesh and northern most Uttar Pradesh, west of Nepal. The Ganges basin includes the entire territory of Nepal and the Uttar Pradesh of India. The river flows in south-easterly direction until it joins the Jamuna at Aricha (Fig 1). The total length of the river from its source to Aricha is about 2200 km and its drainage area is about 980000 km^2. Its W.L. slope near Hardinge bridge is approximately 5 cm/km. The characteristics bed material size D_{50} at u/s and d/s of Hardinge bridge varies from 0.20-0.10 mm and 0.20-0.15 mm respectively (RRI, report Sed-75, 1995/96).

The Padma

Down stream of Aricha the combined Ganges and Jamuna flows are carried by the river known as Padma. It is about 120 km long and joins the upper Meghna at north of Chandpur, after which it is called lower Meghna which debouches into the Bay of Bengal at about 150 km from its confluence at Aricha. Its total drainage area is about 1,600,000 km^2. Its W.L. slope is around 5 cm/km and main distributory is Arial Khan. The characteristics grain size of bed material is somewhat finer to that of the Ganges (RSP, Annex-3, 1996). Network of main river system of Bangladesh with gauging station has been shown in **Fig 1** as given below.

Methodology

Tests were carried out on the sediment samples of the three main rivers using standard methodology. The grain size distribution tests were carried out on the samples by Sieve-Hydrometer and Sieve-Pipet methods. In case of sediment classification, MIT standard classification method has been followed. There are various methods for determination of sediment concentrations. In this study Evaporation method has been used for determination of sediment concentration.

Laboratory study of the sediment characteristics of the Jamuna

To study the sediment characteristics of the Jamuna, 250 bed material samples and 45 suspended sediment samples collected by BWDB were tested in the Sediment laboratory of RRI. The bed samples were collected from the locations at Sirajgonj and Nagarbari and the suspended sediment samples were collected from Bahadurabad-Fulchuri for 1994 to 1996. The bed samples were analyzed for grain size distribution. Range of percentages of silt content D_{16}, D_{50}, D_{84} and Geometric standard deviation were calculated. The results have been presented in Table 1.1 (RRI, Sed-4, Sed-6 & Sed-7, 1995/96). The suspended sediment samples were tested for determination of concentrations. From these concentration studies the suspended sediment transport rates were calculated and transport rate versus water discharge relationship has been established and shown graphically in **Fig-2** (RRI, Sed-42, Sed-43, 1995/96).

Table 1.1 Silt content, grain sizes and geometric standard deviation of bed material sample of the Jamuna

No of Sample	Location	Percent by Weight		D_{16} (mm)	D_{50} (mm)	D_{84} (mm)	Geometric Standard Deviation
		sand	silt				
164	Sirajgonj	90-100	0-10	0.063-0.20	0.10-0.30	0.16-0.40	1.59-1.41
16	"	60	40	0.05	0.095	0.170	1.84
7	"	85	15	0.067	0.125	0.20	1.73
53	Nagarbari	90-100	0-10	0.063-0.15	0.08-0.25	0.125-0.35	1.42-1.53
6	"	75	25	0.065	0.12	0.19	1.71
4	"	60	40	0.055	0.096	0.175	1.78

Laboratory study of the sediment characteristics of the Ganges

In the sediment laboratory of RRI, 191 bed material and 128 suspended sediment samples collected from the Ganges at Hardinge bridge were analyzed. The samples were collected by BWDB during the year 1995-1997. Tests were carried out on the bed material samples for determining grain size characteristics. Range of percentages of silt and sand content, D_{16}, D_{50}, D_{84} and Geometric standard deviation are calculated from the study of the grain size characteristics. The results have been presented in Table 2.1 (RRI, Sed-75, 1995/96). The suspended sediment samples were tested for determining their sediment concentrations. The suspended sediment transport rate were calculated from the concentrations studies using water discharge supplied by BWDB. A graphical representation of sediment transport rate with water discharge for the period 1995 to 1997 has been shown in **Fig-3**. (RRI, Sed-2, Sed-3 & Sed-14, 1995/96).

Table -2.1 Silt content grain size and geometric standard deviation of bed material samples of the Ganges

No. Of Sample	Location	Percent By Weight		D_{16} (Mm)	D_{50} (Mm)	D_{84} (Mm)	Geometric standard Deviation
		sand	silt				
10	2.5 miles d/s of Hardinge Bridge	90-100	0-10	0.15-0.10	0.20-0.15	0.25-0.18	1.29-1.35
90	U/s of Hardinge Bridge.	90-100	0-10	0.18-0.08	0.20-0.10	0.32-.125	1.35-1.25
25	"	70	30	0.06	0.115	0.175	1.71
66	"	20	80	0.009	0.04	0.08	3.22

Study of the sediment characteristics of the Padma

For studying the sediment characteristics of the Padma 64 suspended sediment samples collected by BWDB from the Mawa station during the period 1994-96 were tested in the sediment laboratory of RRI. Samples were collected during the non-tidal period only and no bed sample was collected during the period. Tests were carried out on the suspended sediment samples for determining their sediment concentrations and grain size distribution. The suspended sediment transport rate were calculated from the concentration studies using water discharge supplied by BWDB. The results of size distribution studies with some characteristics sediment parameters have been shown in Table 3.1 (RRI, Sed-10, 1995/96). The relation between sediment transport rate with water discharge has been shown graphically in **Fig-4** (RRI, Sed-10, 1995/96).

Table-3.1 Silt content, grain sizes and Geometric standard deviation of suspended sediment samples of the Padma.

No. of sample	Location	Percent by weight		D_{16} (mm)	D_{50} (mm)	D_{84} (mm)	Geometric standard deviation
	Mawa	sand	silt				
26	"	50	50	0.045	0.063	0.135	1.77
6	"	75	25	0.060	0.125	0.190	1.80
12	"	35	65	0.030	0.055	0.110	1.92

Discussion

The results presented in table 1.1, 2.1 and 3.1 obtained from the laboratory study of the sediment samples of the Jamuna, the Ganges and the Padma respectively will give the planner and designer an idea about the range of variation of sediment characteristic parameters. It is seen from table 1.1, 2.1 and 3.1 that the characteristic bed material size D_{50} of Jamuna is higher than that of Ganges and the D_{50} of the Ganges is higher than that of the Padma. In the Jamuna at the location of Sirajgonj about 88% of the total samples contains 90 to 100 percent fine sand whereas in the Ganges 84% of total sample contains 90 to 100 percent fine sand. The variations of sediment transport rate with water discharge of the Jamuna at Fulchuri, the Ganges at Hardinge bridge and the Padma at Mawa for consecutive three years have been shown in Figures 2, 3 and 4 respectively. It is seen from Fig. 2, 3 & 4 that the graphs show good relationship between sediment transport rate and water discharge. It is also seen from the figures that the sediment transport rate increases with increase of water discharge. If one variable is known then it is possible to determine other variable from the graph. So for those rivers these may be used as tools for determining one variable when the other is known.

Conclusion

Construction of hydraulic structures like barrage, bridge, irrigation canal, drainage channel, reservoir, closure, regulators etc. are very expensive. Such important and expensive structures do not permit an error in design. A design engineer must have sound knowledge on sediment concentration, sediment transport rate, grain size of the river bed material, bed load material and suspended material for the proper design of structures mentioned above. The sediment characteristic data presented in the paper may help the planners and designers in planning and designing river engineering works within the domain of the rivers. From the study it can be concluded that the D_{50} of Jamuna is higher than that of the Ganges and the D_{50} of the Ganges is higher than that of the Padma. It may also be concluded that the geometric standard deviation increases as the silt content increases. So, the higher value of geometric standard deviation indicates the higher percentage of silt content in the sample. From the sediment transport-water discharge relationship it can be concluded that if one variable is known then the other variable can be found. Since a good knowledge on sediment characteristics is very essential for better understanding of sedimentology and morphology of the rivers. Hence for detail information about the sediment characteristics of the main rivers, more sediment samples should be collected from the different locations of the rivers and elaborate laboratory studies should be carried out on the samples by sophisticated equipment.

Acknowledgement

The authors like to express their gratitude to the field personnel of the Hydrology Directorate of BWDB for collection of sediment samples and supply of field data. They also like to thank sediment technicians and other RRI personnel those who were associated with ttestings of the samples.

References

ASCE (1977) **Sedimentation Engineering,** publised in Reports on Engineering Practice No. 54.

Jansen, P. Ph. & Others ed. (1979) : **Principle of river engineering, The Netherlands.**

River Survey Project, FAP-24 (1996) **Bed material sampling in the Ganges, Padma and Jamuna**, Special Report no. 8, FPCO, GoB, Dhaka.

River Survey Project, FAP-24 (1996) **Sedimentology,** Final report, Annex-4, FPCO, GoB, Dhaka.

River Survey Project, FAP-24 (1996) **Hydrology**, Final report, Annex-3, FPCO, GoB, Dhaka.

Sedimentation Methods Manual (1986) Delft Hydraulic Laboratory Delft, The Netherlands.

Sediment Testing Report RRI (1994-1997) **Analysis of sediment samples of Ganges, Jamuna and Padma,** Report No. Sed-2, Sed-3, Sed-4, Sed-6, Sed-7, Sed-10, Sed-14, Sed-42, Sed-43 & Sed-75.

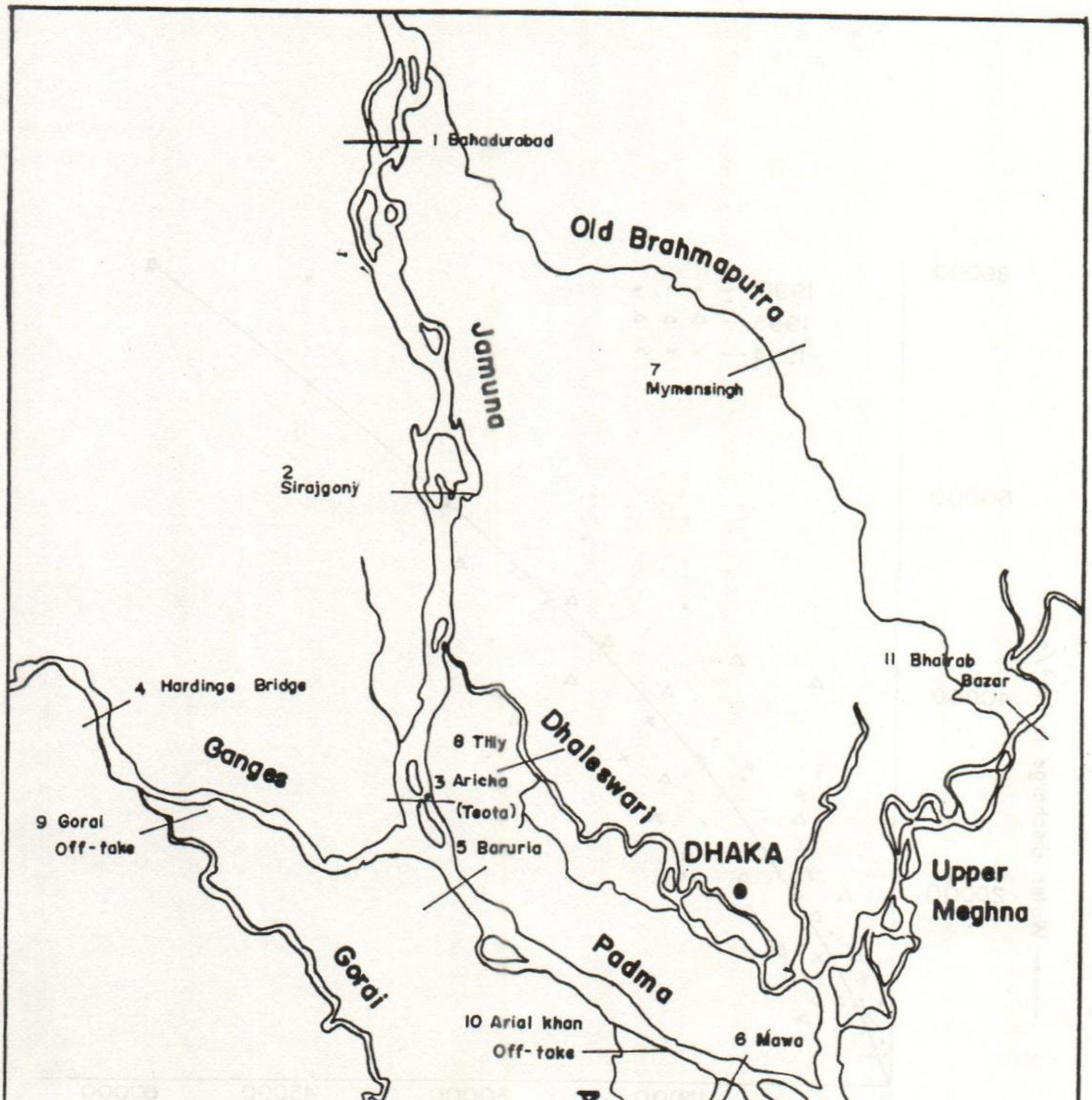
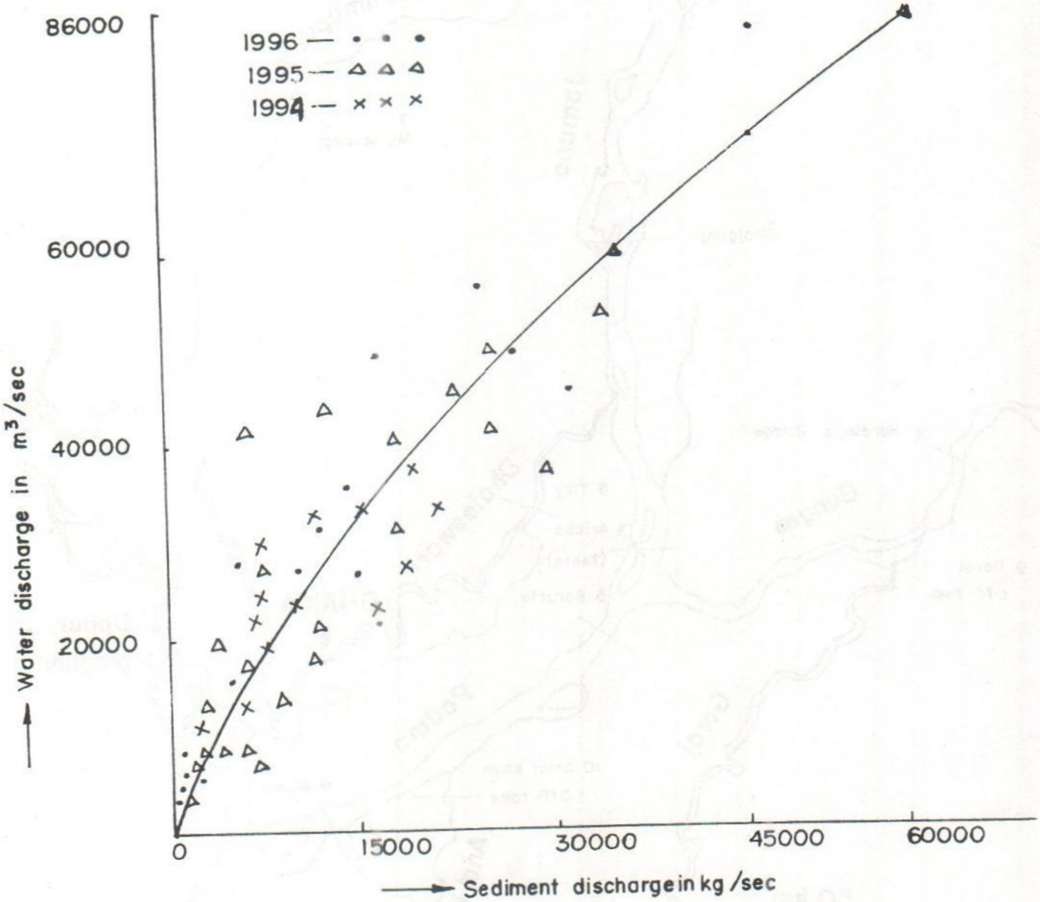


Fig :- I Network of main river system of Bangladesh with gauging stations.



Fig—2 Jamuna at Bahadurabad - Fulchuri.

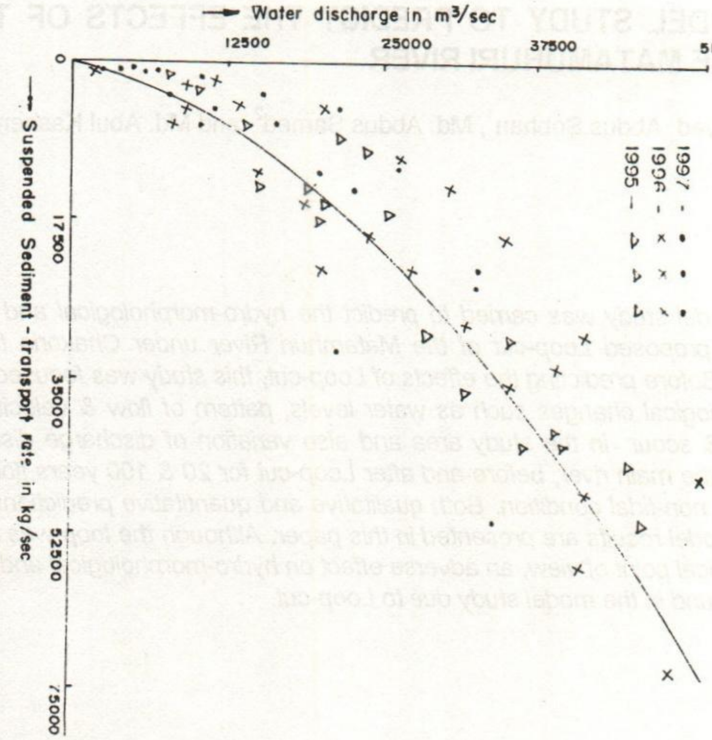


Fig – 3 Ganges at Hordinge bridge

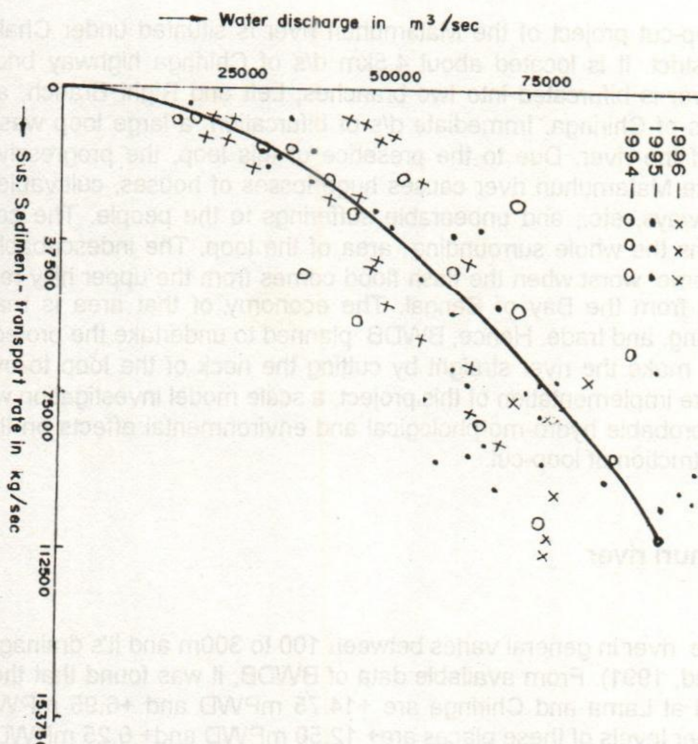


Fig 4 Padma at Mawa

SCALE MODEL STUDY TO PREDICT THE EFFECTS OF THE LOOP-CUT OF THE MATAMUHURI RIVER

Syed Abdus Sobhan[1], Md. Abdus Samad[2] and Md. Abul Kashem[3]

Abstract

A Physical model study was carried to predict the hydro-morphological and environmental effects of the proposed Loop-cut of the Matamhuri River under Chakoria thana of Cox's Bazar district. Before predicting the effects of Loop-cut, this study was focused mainly on the Hydro-morphological changes such as water levels, pattern of flow & velocity, substantial bank erosion & scour in the study area and also variation of discharge distribution at the bifurcation of the main river, before and after Loop-cut for 20 & 100 years flood frequencies and for tidal & non-tidal condition. Both qualitative and quantitative predictions made on the basis of the model results are presented in this paper. Although the loop was matured to cut from the technical point of view, an adverse effect on hydro-morphological and environmental system was found in the model study due to Loop-cut.

Introduction

Matamuhuri loop-cut project of the Matamuhuri river is situated under Chakaria Thana of Cox's Bazar district. It is located about 4.5km d/s of Chiringa highway bridge. The main Matamuhuri River is bifurcated into two branches, Left and Right Branch, at Betua Bazar about 4.0km d/s of Chiringa. Immediate d/s of bifurcation, a large loop was formed in the Right Branch of the river. Due to the presence of this loop, the progressive erosion and inundation of the Matamuhuri river causes huge losses of houses, cultivable land & crops, roads and highways, etc., and unbearable sufferings to the people. The continuous bank erosion threatens the whole surrounding area of the loop. The indescribable sufferings of the people become worst when the flash flood comes from the upper hilly region meet with the tide comes from the Bay of Bengal. The economy of that area is mainly based on agriculture, fishing, and trade. Hence, BWDB planned to undertake the protective measures and decided to make the river straight by cutting the neck of the loop to overcome these problems. Before implementation of this project, a scale model investigation was carried out to predict the probable hydro-morphological and environmental effects on the project area due to the construction of loop-cut.

The Matamuhuri river

The width of the river in general varies between 100 to 300m and it's drainage area is about 195 km^2 (Wazed, 1991). From available data of BWDB, it was found that the highest water levels recorded at Lama and Chiringa are +14.75 mPWD and +6.95 mPWD respectively where the danger levels of these places are+ 12.50 mPWD and+ 6.25 mPWD respectively.

---

[1]Director General, RRI, Faridpur; [2]Director (In Charge), Hydraulic Research, RRI, Faridpur
[3]Scinetific Officer, RRI, Faridpur

The lowest water levels recorded at Lama and Chiringa are +5.23 mPWD and +1.80 mPWD respectively. The maximum peak discharge observed at Lama is about 4750 m^3/s in 1988. The minimum discharge observed at Lama is about 1.12 m^3/s in 1976. The slope of the Matamuhuri river varies from about 12 to 35 cm/km. The bed material of Matamuhuri consists of fine sand with median diameter of about 0.081mm.

Objectives of the model study

The main objective of the model study to predict the effect of loop-cut are:

- To identify the possible locations of erosion and deposition in the modeled area due to the construction of loop-cut
- To investigate the variation in water level, velocity at u/s & d/s of loop cut and cross-dam.
- To find out the probable back water effects due to the embankment to be constructed
- To furnish design parameters for finalization of the river training works.

Methodology of the study

The model was operated for discharge of 20 & 100 years return period to investigate the hydro-morphological parameters of the study reach for both non tidal and tidal condition. The size of the bed material in the model was not possible to scale down. The size of bed material used in the model is almost similar to that of the prototype. To overcome this constraint and to simulate the scour processes, the procedure of FAP 21/22, 1993 was followed to select the scour discharge. But in this case, the Froudian design discharge was found sufficient for scour simulation where the average velocity in the model was about 1.5 times greater than critical velocity for sand movement.

Assessment of basic data

Discharge data recorded at Lama and Water levels data recorded at both Lama and Chiringa were collected from BWDB. Bathymetry survey of the Matamuhuri river at project area was done by River Research Institute (RRI) with the help of BWDB. Bed and bank material samples were also collected during the bathymetry survey and were analyzed at RRI.

A frequency analysis was carried out on consistent data only. The Gumbel EV1 distribution was selected for the analysis of extreme values i.e. yearly maximum discharges and water levels. The discharge and water levels for the return periods of 20 years and 100 years were estimated by the method of moments [Shahin et al, 1993].

The discharges and water levels for different return periods are as follows:

Frequency	Discharge at Lama (m^3/s)	Water level at Lama +mPWD	Water level at Chiringa +mPWD
1:20	2546	14.67	7.354
1:100	3343	16.475	8.286

The distance of Chiringa from Lama gauge station is about 29km. The calculated water surface slopes of the Matamuhuri River corresponding to 100 and 20 years flood frequency were about 28 cm/km and 25cm/km respectively. As there is no record of discharge at Chiringa, so from the consideration of the physical condition of Matamuhuri River system, about 90% of 100 & 20 years flood frequency (discharge) of the main river at Lama was taken as the design discharge for the model study at upstream of the Bifurcation (C/S-3) (n). It is assumed that about 10% of the total discharge overflows the upstream bank and flows in other way. These discharges are almost same as the calculated discharge from the Chezy's equation. For example, calculated discharge corresponding to the water level for a returned periods of 20 years is about 2277 m3/s which almost equal to the 90% of the discharge at lama. Hence, the estimated design discharges and water levels at C/S-3 in the main river upstream of the bifurcation point are:

Frequency	Discharge at C/S-3(main river) (m^3/s)	Water level at C/S-3(main river) +mPWD
1:20	2300	6.4
1:100	3000	7.21

The water levels at C/S-26 which was considered as the downstream boundary between polder nos. 65 & 65/A-3. For the determination of water levels at C/S-26 of the Right Branch, the water surface slope were selected as 20 cm/km and 35 cm/km for loop and loop-cut condition respectively. The corresponding water levels at C/S-26 for different return periods are:

Frequency	Water level at C/S-26(Right Branch) with loop +mPWD	Water level at C/S-26(Right Branch) with loop-cut +mPWD
1:20	4.98	5.35
1:100	5.79	6.15

The discharge distribution at any branch such as Left branch or Right branch at Bifurcation could be estimated from the following relationship by using Chezy's Equation and by assuming S & C constant [Brueses, 1987];

$$Q_L = Q_T * (A_L/A_T) * \sqrt{(D_L/D_T)}$$

where; Q_L= Discharge for left branch, Q_T= Discharge for the main river, A_L= Cross-sectional area of left branch, A_T= Cross-sectional area of the main river, D_L= Equivalent water depth for the left branch, D_T= Equivalent water depth for the main river, S= slope, C= Chezy's Co-efficient.

From the above relationship, it was found that about 45% of the total discharge of the Matamuhuri main river flowing through the left branch of the river i.e. Q_L=.45Q_T. Hence, this criteria was also used as lateral boundary condition for model calibration .Moreover, from the relation of Q_L=.45Q_T , it is also clear that about 55% of the total discharge of the main river flows through the right branch of the river.

Loop maturity

The length of the loop (bend)is about 5300 m where the length of the chord channel is about 730m. So the cut-off ratio is about 7 which is greater than threshold no. for cut-off as 3 (Garg,1986) Again the radius of the curvature is about 950 m where the maximum discharge at the modeled area is about 4275 m^3/s (90% of max. discharge 4750 m^3/s at Lama). So, the ratio r/Sqrt(Q_{max}) is about 14 which is between threshold limits of 13 to 24 (Garg,1986). Hence from the technical point of view, the loop is matured for cutting.

Model design and construction

The Model was designed according to the Froude's model law. Considering the physical facilities of RRI and for the simulation of the prototype behavior, an undistorted model scale of 1:70 was selected to ensure sufficient depth for precise measurement & sediment movement and to fulfill rough turbulent flow in the model. The model design parameters along with scale factor are given in Table 1.

Table 1: Model design parameters and model scale ratios

Parameters	Unit	Prototype	Model	Scale factor
Length	m	7000	100	70
Width	m	240	3.43	70
Mean depth	m	4.35	0.062	70
Discharge	m^3/s	3000	73.18	40996
Average flow velocity	m/s	2.87	0.343	8.367

(Source: RRI, 1998)

About 8 km river reach including loop-cut & a portion of left branch covering the total width and about 2 km river reach at d/s of the loop-cut was reproduced in the model. The Harbung khal was also reproduced in the model. To supply steady state flow in the model, necessary arrangements were made in the model area. A water circulation system was set-up for uninterrupted water supply in the model (Fig. 1). To maintained inflow, a rectangular sharp crested was installed at upstream of the model. Also, to provide tidal flow in the model, another sharp crested weir was installed at downstream of the model. Two well gauges were installed to record water levels in the model. Two tail gates were fixed at the downstream end of the model to maintain required water level. Provision was made for watering and dewatering of the model bed before and after test runs.

Boundary conditions

The upstream boundary condition was the discharge distribution at C/S-3 of the main river at the upstream limit of the model and also at upstream of the bifurcation. The downstream boundary condition was the water level at the C/S-26 of right branch at the downstream end of the model. The lateral boundary condition was the discharge distribution at C/S-2 of left branch at downstream of the bifurcation and the discharge of left branch was 1350 m^3/s which is about 45% of the discharge of the main river.

Measurements

Discharge was determined from the water level measured at upstream of a standard rectangular sharp crested weir installed at the upstream end of the model. The water level (h_w) was standardized for direct measurements of the discharges according to Rehbock's formula [RRI-BUET, 1997] and it is:

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

where, g= acceleration due to gravity (m/s^2); h_w= vertical distance between the crest of weir and the upstream water level (m); p= height of the crest of the weir above the bottom (m); Q= Discharge passing the weir (m^3/s); U_w= average flow velocity in the cross section in which h_w is measured (m/s); b=width of the weir.

The water levels were measured from the gauge readings throughout the model bed. Time averaged flow velocities were measured at 0.2 and 0.8 depth with A-ott current meters along the cross sections at a regular interval. The surface velocity at shallow depths were measured by floats and to obtain the mean surface velocity was multiplied by a factor of 0.85 formula [RRI-BUET, 1996]. The slope of water surface in the model was determined from water levels registered at upstream and downstream gauges. Float tracking was performed at different cross sections for determining flow lines. The scour depths were measured by inserting a meter scale into the scour hole with respect to a fixed reference line whose R.L. was known

Model calibration

Three boundary conditions were satisfied during the calibration test. The C/S-3(main) u/s of the bifurcation was selected as a representative inflow section. The model was calibrated by several trial runs. The model was calibrated against design discharge of 3000 m^3/s. The calibration results are presented in the Table nos. 2, 3, & 4.

Table-2: Comparison of parameters between prototype and model at C/S-3 (main river)

Parameter	Unit	Proto-type data	Observed value in model	Scale factor	Corresponding Prototype value
Discharge	m^3/s	3000	0.0738	40996	3025
Mean depth	m	4.35	0.0628	70	4.40
Ave. flow velocity	m/s	2.87	0.343	8.367	2.87

(Source: RRI, 1998)

Table-3: Comparison of the water level of prototype with the measured water level in model at C/S-26 (right) and Q=3000 m^3/s

Prototype water level in +mPWD	Measured water level in model +mPWD
5.79	5.79

(Source: RRI, 1998)

Table-4: Comparison of the discharge distribution of the left branch in prototype with the measured discharge in the model at C/S-2(left).

Parameter	Unit	Prototype Data	Observed value in model	Scale factor	Corresponding Prototype value
Discharge	m^3/s	1350	0.0329	40996	1340
Q_{left}/Q_{main}	%	45%	44.7%	1	44.7%

Note: Q_{main}=Discharge of the main river= 3000^{m3}/s (Source: RRI, 1998)

Test runs

The model study was carried out with embankment where no embankment were existed except a few polder existed at the d/s of the model area. Several tests were performed for the discharges of returned periods of 100 and 20 years for different options to investigate the variation in water level, velocity at u/s & d/s of loop cut and cross-dam as well as furnishing the parameters of the river training works.The model was run until the dynamic equilibrium was reached.

Findings

In different test scenarios, flow velocity, float tracking, bank erosion and bed scour level were recorded. Hence, the detail analysis of the results of the physical model test for both 100 & 20 years flood frequency are the basis for the findings (general and detailed) are given here:

General findings

(A) Without Loop-cut and with Embankment

(i) The study was conducted on embanked condition of the river system where no embankment existed in the prototype i.e., in real field. So higher water level was observed due to embankment before loop-cut (+9.00 mPWD and +8.49 mPWD in nontidal for 100 and 20 years flood frequency respectively), and it was estimated about 2m higher than the normal design condition(+7.21 mPWD and +6.4 mPWD for 100 and 20 years flood frequency respectively) at C/S-3 (main river) u/s of bifurcation. Severity of erosion was found within the loop.

(B) With Loop-cut, Embankment and Cross-dam

(i) The tidal ranges of 0.3m and 0.4m for 100 and 20 years flood frequency respectively were observed at C/S-3(main) u/s of the bifurcation and of 0.50 m and 1.5 m for 100 and 20 years flood frequency respectively at C/S-26(right) d/s of the loop-cut.

(ii) Due to loop-cut, water levels were reduced by about 0.4m & 0.24m at C/S-3 (main) river at u/s of bifurcation for 100 and 20 years flood frequency respectively. But the net rise in water levels due to embankment are about 1.39 m and 1.85 m above design water levels and may be extended to about 1.0 km & 3.0 km u/s of Chiringa for 100 and 20 years flood frequency respectively.

(iii) The intensity of erosion and inundation increases in modeled area with the increase of tide.

(iv) The erosion at both banks of the loop were reduced significantly and a dead water storage at d/s (behind) of the cross-dam was observed. The water level at d/s of the cross-dam was observed as about +7.00 mPWD and +6.8 mPWD for 100 and 20 years flood frequency respectively which may be created temporary inundation if embankment does not exist.

(v) High intensity of bank erosion was observed within and at d/s of the loop-cut as well as at the toe of the hill locates near the loop-cut. The char land at d/s of the loop-cut would be eroded gradually. But a shifting tendency of the left bank between bifurcation point and the u/s end of the loop cut was observed.

(vi). Due to the loop cut with cross dam, it was observed that the discharge at the left branch was about 25-30% of the total discharge where the discharge at the loop-cut was about 70-75% of the total discharge.

(vii). Due to loop-cut without cross dam, the u/s water level at C/S-3 (main) was reduced by about 1.0 m for both 100 & 20 years flood frequency but the erosion would be occurred in the loop. The bank of u/s right corner of the loop-cut would be shifted towards right due to high erosion. A large sand bar might be formed gradually near u/s left corner and d/s right corner of the loop-cut.

(viii) The boulder sizes proposed by BWDB was found stable against the velocity about 4.0 m/s.

(ix) The water level difference between the u/s and d/s of the cross-dam is about 1.4 m observed in the model for both 100 & 20 years flood frequency.

(x) The loop-cut would become in natural shape after flood. The single slope between the ground (+5.5 mPWD)and the bottom of the loop-cut would be better.

(xi) The smoothed u/s left corner of loop-cut with cross-dam might be reduced the erosion on u/s right corner of the loop-cut.

Detail findings

Option-1; Condition: Loop-cut, Cross-dam, Embankment and 100 years flood frequency

Water levels [excluded tidal range at C/S-3(main) about 0.3m and at C/S-26(right) is about 0.5m]:

At C/S-3 of the main river u/s of bifurcation	= +8.60 mPWD
At C/S-26 of the right branch d/s of loop-cut	= +6.15 mPWD
At the u/s end of the loop-cut	= +8.00 mPWD
At the d/s end of the loop-cut	= +7.35 mPWD
At the u/s of the cross-dam	= +8.40 mPWD
At the d/s of the cross-dam	= +7.00 mPWD

The locations of erosion and deposition observed in the model are shown in Fig. 2. It may be noted that the erosion may be extended further down stream of C/S-26(right). The locations of erosion would require protective measure.

Option-2; Condition: Loop-cut, Cross-dam, Embankment and 20 years flood frequency

Water level [excluded tidal range at C/S-3(main) about 0.4m and at C/S-26 (right) about 1.5m]:

At C/S-3 of the main river u/s of bifurcation	= +8.25 mPWD
At C/S-26 of the right branch d/s of loop-cut	= +5.35 mPWD
At the u/s end of the loop-cut	= +7.80 mPWD
At the d/s end of the loop-cut	= +6.80 mPWD
At the u/s of the cross-dam	= +8.20 mPWD
At the d/s of the cross-dam	= +6.80 mPWD

The locations of erosion and deposition observed in the model are shown in Fig. 3. It may be noted that the erosion may be extended further down stream of C/S-26(right). The locations of erosion would require protective measure.

Option-3; Condition: Loop-cut, Embankment without Cross-dam and 100 & 20 years flood frequency

Water levels for 100 years flood frequency [excluded tidal range at C/S-3(main) about 0.3m and at C/S-26 (right) about 0.5m]

At C/S-3 of the main river u/s of bifurcation	=+7.90 mPWD
At C/S-26 of the right branch d/s of loop-cut	=+5.79 mPWD
At the u/s end of the loop-cut	=+7.49 mPWD
At the d/s end of the loop-cut	=+7.30 mPWD

Water levels for 20 years flood frequency [excluded tidal range at C/S-3 (main) about 0.4m and at C/S-26 (right) about 1.5m]:

At C/S-3 of the main river u/s of bifurcation	=+7.30 mPWD
At C/S-26 of the right branch d/s of loop-cut	=+4.98 mPWD
At the u/s end of the loop-cut	=+7.14 mPWD
At the d/s end of the loop-cut	=+6.70 mPWD

The discharge distributions found in the model are as follows:

Return Periods in Years	Discharge in Main River (Q_{main}) in M^3/S	Discharge in Left Branch (Q_{left}) in M^3/S And Corresponding % of Discharge of Main River	Discharge in Right Branch (Q_{right}) in M^3/S And Corresponding % of Discharge of Main River	Discharge in Loop-Cut (Q_{loop}) in M^3/S And Corresponding % of Discharge of Main River
100	3000	810 & 27%	690 & 23%	1500 & 50%
20	2300	598 & 26%	575 & 25%	1127 & 49%

(Source: RRI, Report no. 126, January, 1998)

For this option, the detailed erosion and deposition pattern found in the model study shown in Fig. 4. The bank of u/s right corner of the loop-cut would be shifted towards right due to high erosion. Also, a large sand bar might be formed gradually near left u/s corner of the loop-cut which would require re-excavation.

Moreover, for these three options, findings about Revetment, Embankment and Loop-cut are as follows:

Revetment specification:

Parameters	Specifications of Revetment
Top level	+5.5 mPWD
Length of apron	9 m
Side slope	1:2
Boulder size	35-40cm (30%), 30-35cm (50%), 25-30cm (20%)

The boulder was found stable against maximum velocity of about 4.0 m/s. Since the top level of the revetment is +5.5 mPWD as the ground level which is less than the water levels found in the model study, so a careful attention should be taken during construction to protect seepage failure. Otherwise the top level of the revetment should be raised.

Embankment and loop-cut

The top levels of the embankments was found satisfactory from +9.0 to +8.5 mPWD for u/s river reach and loop-cut. For the d/s river reach of the loop cut, the top level of embankment was found satisfactory from +8.0 to +7.5 mPWD. The bottom levels of the loop-cut were found about from +3.0 to + 2.0 m PWD after deposition where the design bed levels were given in the model from +1.0 to +0.0 mPWD. But embankments on both banks should be according to the water levels furnished above. Some deposition might require re-excavation.

Conclusion

The scale model study of the Loop-cut of the Matamuhuri river under Chakoria thana of Cox's Bazar district was conducted based on the bathymetry of February-March,1997 to predict the effects of the loop-cut after its construction. The main limitation of the study was that the alignment and width of the loop-cut had to keep fixed as proposed by BWDB due to the presence of the two hills on both sides of the proposed loop-cut. However, on the basis of the findings of the model study, the concluded predictions are as follows.

Despite the loop is matured to cut from the technical point of view, an adverse effects on hydro-morphological and environmental system will be found due to the construction of Loop-cut at Right Branch. The discharge of the Left Branch of the Matamuhuri River at d/s of bifurcation at Betua Bazar (about 4 km downstream of Chiringa Road way bridge), will be drastically reduced due to the Loop-cut at Right Branch which will also cause shortage of water in surrounding areas during dry season. The severe erosion will be occurred at upstream, within and downstream of the Loop-cut as well as at the toe of the adjacent hills which will be created severe environmental degradation of the surrounding hilly areas and as a consequence considerable re-excavation at down stream and within loop-cut would also be required until the loop-cut becomes a natural channel. Back water effect would be extended to about 3km u/s of Chiringa due to the Embankment to be constructed.

Acknowledgment

The authors acknowledge to all concerned officer and staff of RRI & BWDB who helped directly and indirectly to carry out the model study.

Abbreviation

BWDB	Bangladesh Water Development Board
D/S	Downstream
RRI	River Research Institute
U/S	Upstream

References

Breusers, H.N.C.(1987) **River And River Hydraulics**, Class Note, International Institute For Infrastructural, Hydraulic and Environmental Engineering [IHE], Delft, The Netherlands.

Garg, S.K. (1986) **Irrigation Engineering and Hydraulic structures**, India.

M. Shahin H.J.L. Van Oorschot and S.J. De Lange (1993) **Statistical Analysis in Water Resources engineering**, A. A. Balkema Publishers, Holland.

RRI-BUET (1996) **Interim Report No.1**, Scale Model Study of Jamuna River at the Jamuna Bridge Project Area.

RRI-BUET (1997) **Interim Report No.2**, Scale Model Study of Jamuna River at the Jamuna Bridge Project Area, (Sectional Model on East Guide Bund).

RRI (1998) **Report no.126**, Scale Model Investigation for the Loop-cut of the Matamuhuri River under Cox's Bazar District.

Wazed, M.A. (1991) Rivers of Bangladesh.

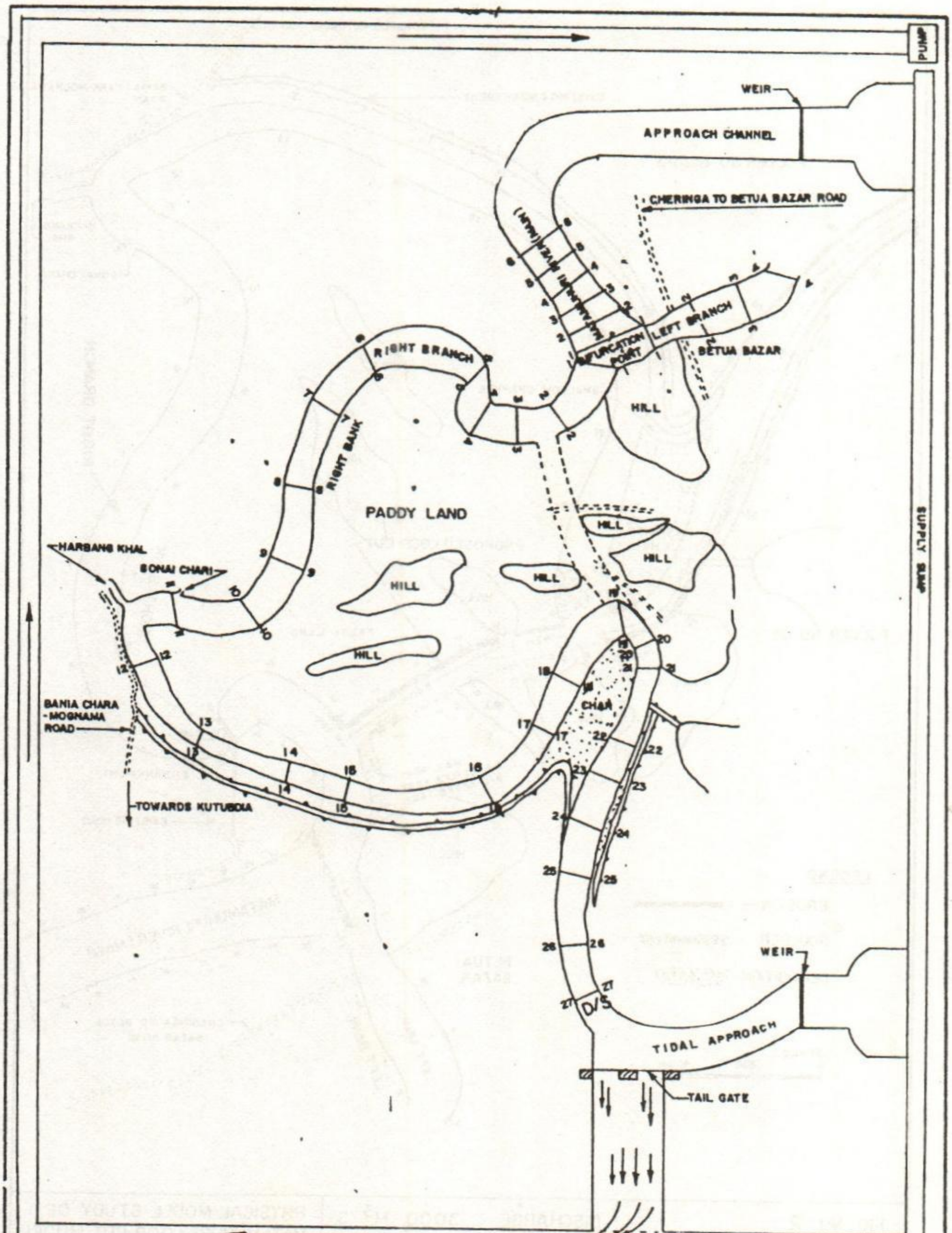


FIG. NO. 1 : MODEL LAYOUT AND WATER CIRCULATION SYSTEM

PHYSICAL MODEL STUDY OF MATAMUHURI LOOP -CUT MODEL

R R I, FARIDPUR

SOURC: RRI, REPORT NO. 126, JANUARY, 1998

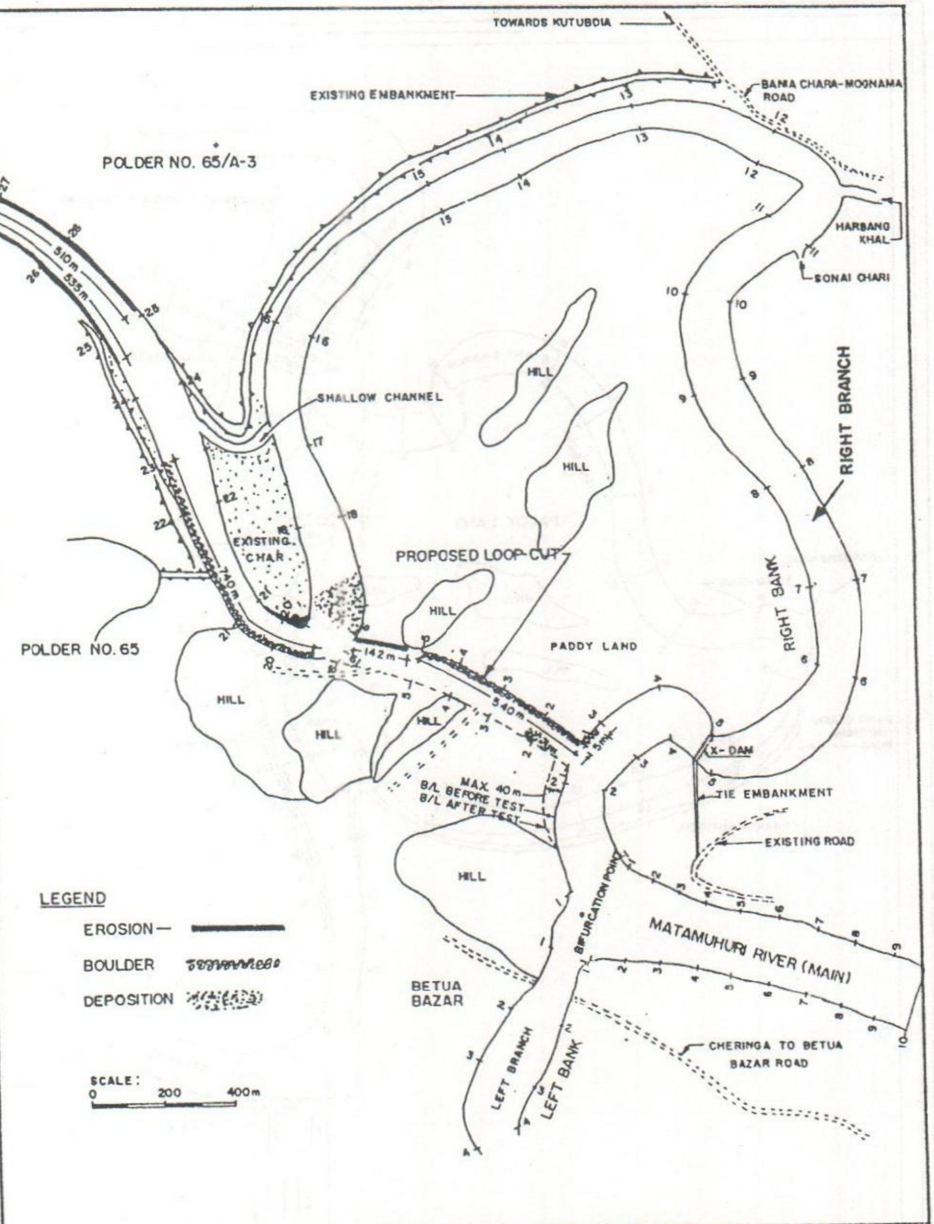


FIG. NO. 2	DISCHARGE : 3000 M^3/S	PHYSICAL MODEL STUDY OF MATAMUHURI LOOP-CUT MODEL
OPTION- I	RETURN PERIOD : 100 YEARS	R R I, FARIDPUR

SOURCE: RRI, REPORT NO. 126, JANUARY, 1998

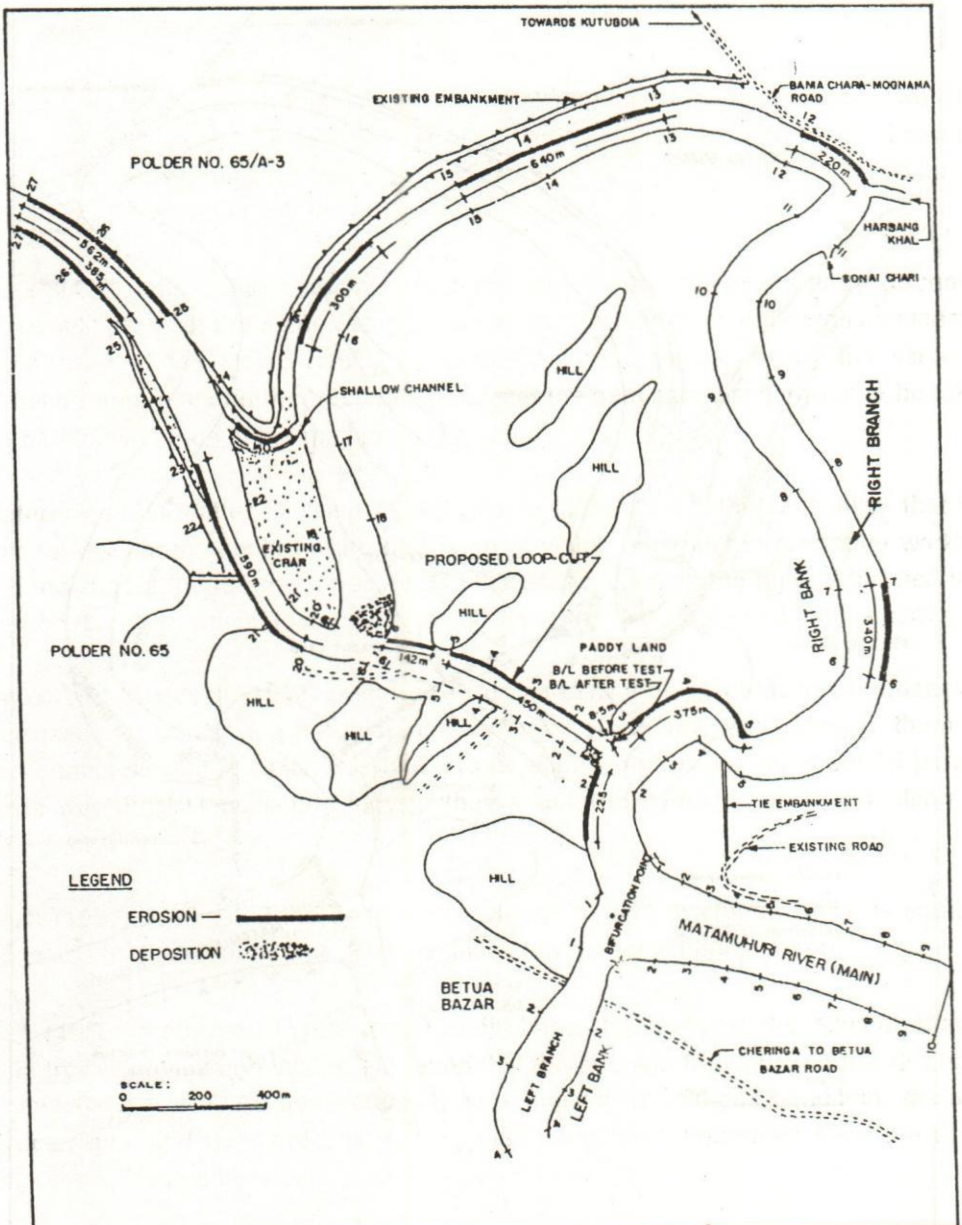


FIG. NO. 3 OPTION-2	DISCHARGE : 3000 M^3/S RETURN PERIOD : 100 YEARS	PHYSICAL MODEL STUDY OF MATAMUHURI LOOP-CUT MODEL R R I, FARIDPUR

SOURCE : RRI, REPORT NO.126, JANUARY, 1998

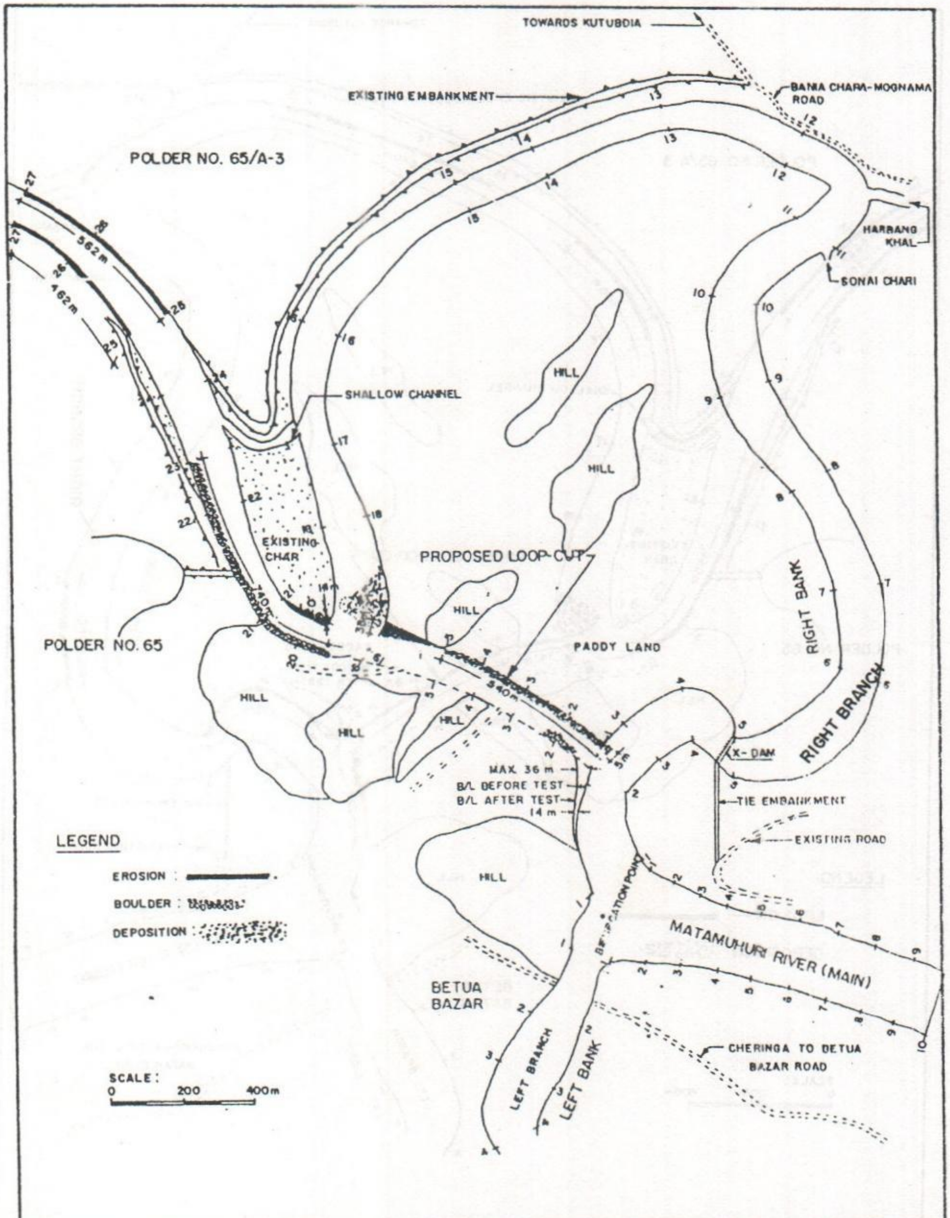


FIG. NO. 4	DISCHARGE : 2300 M^3/S	PHYSICAL MODEL STUDY OF MATAMUHURI LOOP-CUT MODEL
OPTION- 3	RETURN PERIOD : 20 YEARS	R R I, FARIDPUR

SOURCE : RRI, REPORT NO. 126, JANUARY, 1998

EFFECTIVENESS OF REINFORCED CEMENT CONCRETE SPUR CONSTRUCTED ON THE FLOOD PLAIN

Md. Abdus Samad[1], Swapan Kumar Das[2], A.K.M.Ashrafuzzaman[3], Syed Md. Mymur Sultan[3] and Md. Manjurul Haque[3]

Abstract

It is common practice to construct spur into the river extended from the bank. But in the major rivers sometimes it is very difficult to construct spur into the river as per design, as a result constructed spurs can not function properly. Construction of spur on the flood plain is much more easier and even more cost effective. Effectiveness of impermeable Reinforced Cement Concrete(RCC) spur on the flood plain was investigated by physical modelling as a case study. The outcome of this study could also be applied for other rivers and should be optimized by physical model investigation.

Introduction

The study was carried out on the protection of Chandanbisa Bazar from the erosion of the Jamuna river. Chandanbaisa Bazar on the right bank of the River Jamuna under Sariakandi Thana in the District of Bogra was threatened due to continuous bank erosion, which may also endanger the stability of the Brahmaputra Right Embankment (BRE). Bangladesh Water Development Board (BWDB) planned to undertake the protective measures for the Chandanbaisa Bazar, and accordingly BWDB proposed a tentative design of reinforced cement concrete (RCC) spur to save this area with important installations, valuable lands and homestead.

Generally spurs are constructed from the bank into the river for bank protection and river training works. It is always observed that if the length of the spur is too large to be constructed into the river it becomes very difficult to do it as per design. So if situation permits it is better to construct spur on the flood plain which may be more effective and less costly. A physical model study was undertaken to investigate the effectiveness of impermeable RCC spur to be constructed on the flood plain for the protection of Chandanbaisa area from the erosion of the Jamuna river.

---

[1]Director (In Charge), Hydraulic Research, RRI, Faridpur
[2]Senior Scientific Officer, RRI, Faridpur
[3]Scientific Officer, RRI, Faridpur

A total of 3.8 km river reach from cross section 26 to cross section 7 from upstream to downstream and average river width of 1.6 km including BRE at the right were reproduced in the model considering pump capacity and available space to accommodate the model. Index map of Jamuna river showing the study area is shown in Fig-1.

Literature review

The spurs are stone, gravel, rock, earth or pile structures at an angle to a river bank to deflect flowing water away from the critical zones, to establish a more desirable channel for flood control, navigation and erosion control etc. (Richardson, 1975). A spur, pointing upstream called repelling spur, has the property of repelling the flow away from it and spurs, pointing downstream called attracting spur, has the property of attracting the flow towards it (Joglekar, 1971). In repelling spur, scour holes caused by the formation of vertical eddies are developed away from the bank whereas in an attracting spur, scour holes are developed nearer the bank (Garg, 1987). Since an attracting spur brings the water current as well as the scour holes nearer the bank, and makes it more susceptible to damage and hence, are not generally used. The spurs are, therefore, generally aligned either perpendicular to the bank or pointing upstream. The former is generally used on convex banks and the latter on concave banks. When the length of an upstream-pointing spur is small such that it changes only the direction of flow without repelling it, it is called a deflecting spur, instead of calling it a repelling spur. The repelling spurs are generally found to serve the desired results, provided they are properly located with due regard to their position in relation to the meander length. It is desirable to test their performance in hydraulic models before constructing them in actual field. The angle of deflection upstream, is about 10 degree to 30 degree, with a line normal to the bank. In the study of Sastry, 1962, it is shown that a spur angled upstream produce greater scour depth than those normal to the flow direction and a spur angled downstream caused the least depth of scour. Spurs may be constructed either singly or in series, depending upon the need. When constructed in series, they are more effective as they create a pool of almost still water between them which resists the current and gradually accumulates silt between them, thus forming almost a permanent bank after certain time. The choice of using them in series arises, if the reach to be protected is long or if a single spur is neither strong enough to deflect the current nor effective for silt deposition at upstream and downstream of itself (FAP-21/22, 1993).

Construction of the model

A movable bed model was set up considering 3.8 km river reach and partial width of 1.6 km with an undistorted scale of 1:100. Scales of the model were selected allowing sufficient water depth for precise measurement and ensuring rough turbulent flow and sediment movement in the model. Critical velocity for sand movement in the model and prototype was calculated using the formula (Van Rijn, 1984)

$$V_{cr} = 0.19(D_{50})^{0.1} Log \frac{12h}{3D_{90}}$$

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 $V/V_{cr} > 2$ (FAP- 21/22, 1993). 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.

Model was constructed with fixed bed upstream and downstream and movable bed in the middle. Sharp crested weir was installed at the measuring device upstream of the model to ensure required discharge and tail gates at the downstream to maintain water level in the model. Stilling pond was constructed at immediate downstream of the weir for energy dissipation and hollow bricks were placed at upstream fixed bed for ensuring uniform flow. Water was allowed to pass over the model bed and then over tailgates and finally collected into recirculating sump. Rehbock's formula was used for calculation of discharge over the weir as follows:

$$Q = (0.403 + 0.053\frac{h_w}{p}) \times b\sqrt{2g}\left[\left(h_w + \frac{V_a^2}{2g}\right)^{1.5} - \left(\frac{V_a^2}{2g}\right)^{1.5}\right]$$

Where,

Q = Discharge in m^3/s

p = Height of the weir in m

h_w *= Head over the weir in m*

b = Length of the weir in m

V_a *=Velocity of approach in m/s*

g = Acceleration due to gravity m/s^2

Calibration of the model

For modelling, three boundary conditions were considered as follows :

- An upstream boundary condition at the inflow section of the model
- Downstream boundary condition at the out flow of the model

- Lateral boundary condition is the location and alignment of the imaginary boundary wall constructed in the model as the left boundary for this study

The upstream boundary condition is the discharge distribution along the upstream limit of the model. Discharge distribution at cross section 26 was estimated and compared with measured discharge. Table-1 shows the comparison of computed and measured flow velocities at cross section 26. The downstream boundary condition is the water level which was maintained at the downstream of the model. The lateral boundary is an imaginary line which 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 criteria considered to fix up the lateral boundary were (i) The width of the model should be sufficient to prevent any influence by the spurs on this flow line (ii) The curvature of the boundary wall should be as smooth as possible to avoid flow separation (FAP-21/22, 1993). No influence of the spurs on the left boundary and no flow separation was observed during calibration of the model.

Table-1: Comparison of computed and measured flow velocities at cross section 26

Distance from BRE in m	Computed velocity in m/s	Measured velocity in m/s
752.0	2.33	2.23
880.5	2.8	2.72
1025.9	3.21	3.01
1126.5	3.26	3.18
1226.4	2.31	2.25
1326.7	2.24	2.27
1427.7	2.19	2.10
1527.6	2.14	2.10
1610.0	2.12	2.15

Test runs

Seven test runs were conducted to carryout the study and each test was run until maximum scour was reached. Test no.1 i.e. test with existing conditions, was contributed to calibration of the model. Test no.2 was the only test conducted with four number of spurs (S_1, S_2, S_3 & S_4) proposed by the Design Circle of BWDB extending from existing roads and projected from bank line towards the river (Fig. 3). Test no.3 was same as Test no.2 but the series of spurs were projected from right embankment to bank line on the flood plain. Test no.4 was same as Test no.3 with provision of an additional spur S_5. In Test no.5 the position of S_1, S_2, S_4 & S_5 were same as in Test no.4 but the position of

S_3 was shifted towards upstream by 120 m along bank line with respect to Test no.4 with an additional spur S_6. Test no. 6 was same as Test no. 5. It is noted that Test no.1 to Test no.6 were carried out with discharge of 25,200 cumecs & water level of +16.2 mPWD. In Test no.7 the water level was increased by 1.5 m than the previous test runs keeping the same discharge. The spurs in each test were of equal length (150 m) and perpendicular to bank line and test run was carried out until the equilibrium condition was reached. During each test run near bank flow velocities at different cross-sections, velocity around the spurs, scour depths, water level difference between upstream & downstream of spur etc. have been measured and float tracking was also done.

Interpretation of test results

On the basis of test result from this model study it was found that though some spurs were stable but in some other cases due to higher velocity at the upstream of the spur and may be due to the water level difference between upstream and downstream of the spur, the flow compelled to go through the bottom of the RCC wall. As a consequence soil particles have been washed out because of piping and thus the bottom of RCC wall was exposed to severe scour which affected the spurs. It was also dangerous to the stability of the embankment. Considering velocity and flow lines, Test no.7 was considered the best among all other tests with respect to the performance of the spurs. Maximum net scour measured in this study was 13.1 m and maximum velocity was 2.7 m/s. Near bank velocity measured in different tests are shown in Table-2. It is found from Table-2, near bank velocities were minimum in test no. 7 than that of other tests. Direction of main flow was away from the bank with the spur locations as tested in test no. 7, which is shown in Fig-2. Scour development with time d/s of spur no. 2 where maximum scour was occurred is shown in Fig-3.

Table-2 : Near bank velocity measured in different tests

C/S no.	Near bank velocity in m/s					
	Test no.					
	T1	T2	T3	T4	T5/T6	T7
26	1.7	1.7	1.3	1.2	1.1	1.0
25	2.7	0.0	0.7	0.0	0.0	0.0
24	3.0	0.0	0.5	0.9	-1.0	0.0
23	3.1	0.0	1.2	1.1	0.8	0.7
22	3.2	0.0	0.0	1.3	0.9	0.0
21	3.0	0.0	0.0	0.0	0.0	0.0
20	3.9	0.0	0.0	0.0	0.0	0.0
19	3.4	0.0	2.4	1.8	1.7	1.3
18	2.8	0.0	1.4	2.3	1.6	1.0
17	1.8	0.0	0.0	1.3	0.0	0.0
16	2.5	0.0	0.0	1.7	0.0	0.0
15	2.1	0.0	0.0	0.9	0.0	0.0
14	2.1	0.0	1.2	1.4	0.0	0.0
13	2.7	0.0	-0.6	-0.6	0.0	0.0
12	2.6	1.7	1.9	1.7	1.1	1.0
11	0.8	0.0	0.0	0.0	0.0	0.0
10	2.2	0.0	0.0	2.3	0.0	0.0
9	3.0	0.0	0.7	1.7	1.3	1.2
8	3.7	0.0	1.9	2.3	0.0	0.0
7	3.5	1.9	2.7	2.5	1.6	1.3

Conclusion

It appears from the model study that construction of RCC spur on the flood plain could be effective to protect the Chandanbaisa area. But special attention to be given for the optimization of the design and it could be done from the results of the physical model study. So it may be concluded that if the situation permits, RCC spur construction on the flood plain would be easier and cost effective to serve the purposes. In this study at least first three spurs are suggested to construct and after monitoring the effectiveness in the field rest of the spurs could be constructed.

Acknowledgment

The authors would like to acknowledge all the officers and staff of RRI who were involved in the model study. Special thanks are extended to the concerned Engineers of BWDB for visiting RRI during model test runs and also for their valuable suggestions regarding the model study.

References

Ahmed, M. (1953) **Experiment on design and behavior of Spur-Dike**, Proceedings of IAHR, ASCE, Joint Meeting, University of Minnesota, USA.

FAP 21/22 (1993) **Bank Protection and River Training (AFPM) Pilot Project**, Panning Study, Volume VI, Annex-14.

FAP 21/22 (1992) Bank Protection and River Training Pilot Project: **Physical model tests**, Draft final report, Planning study, Volume VI.

Garg, S.K. (1987) **Irrigation Engineering and Hydraulic Structures**, 7th Edition, Khanna Publishers, New Delhi

Jogleker, D.V. (1971) **Manual on River Behaviour Control and Training**, Publication No.60, Central Board of Irrigation and Power, Page 215-242.

Richardson, E.V. et al (1975) **The Design of Spurs for River Training**, Proceedings of IAHR.

Sharp, J.J. (1981) **Hydraulic Modelling**, The Buttersworth group, Billing and sons, England.

Sastry C.L.N (1962) **Effect of Spur-Dike Inclination on Scour Characteristics**, Thesis Presented to the University of Roorkee, India in partial fulfillment of the requirements for the degree of Master of Engineering.

Van Rijn, C.(1984) **Bed forms and alluvial roughness**, Sediment transport, part-III, ASCE Journal of Hydraulic Engineering, Vol.110, No.12, pp.1433,1434,1736 &1750.

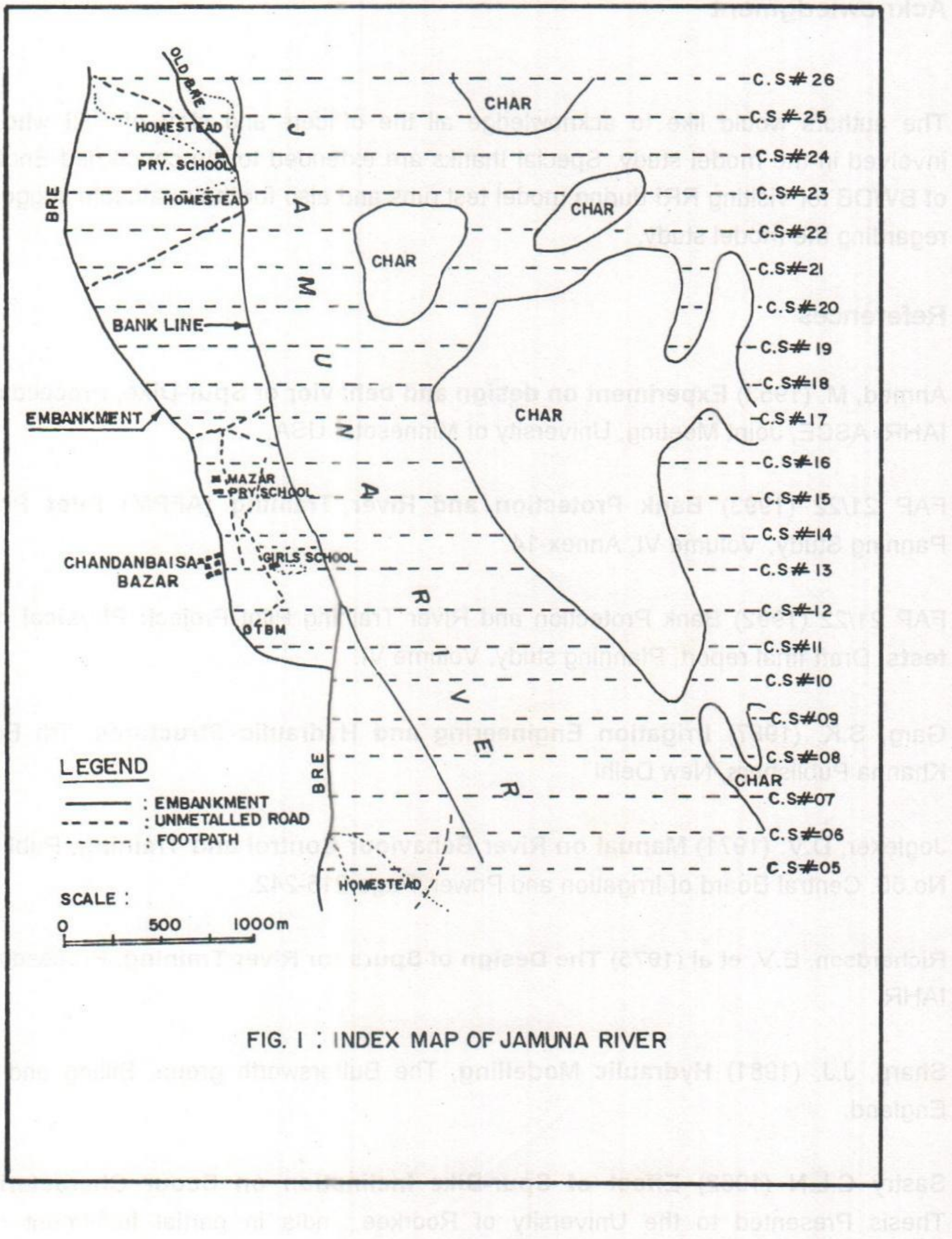


FIG. I : INDEX MAP OF JAMUNA RIVER

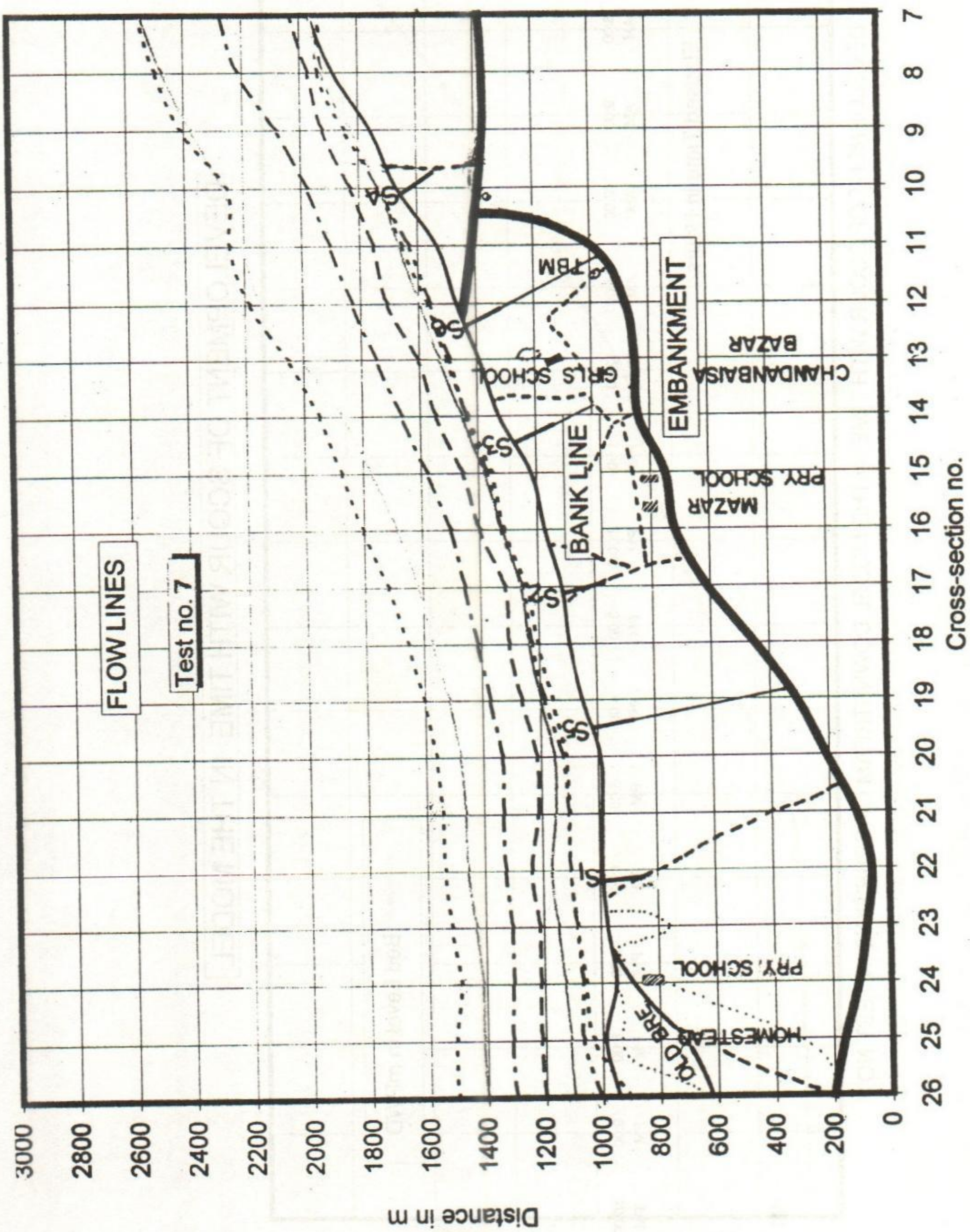


FIG-2 : FLOW LINES AND LOCATION OF SPURS IN RECOMMENDED TEST

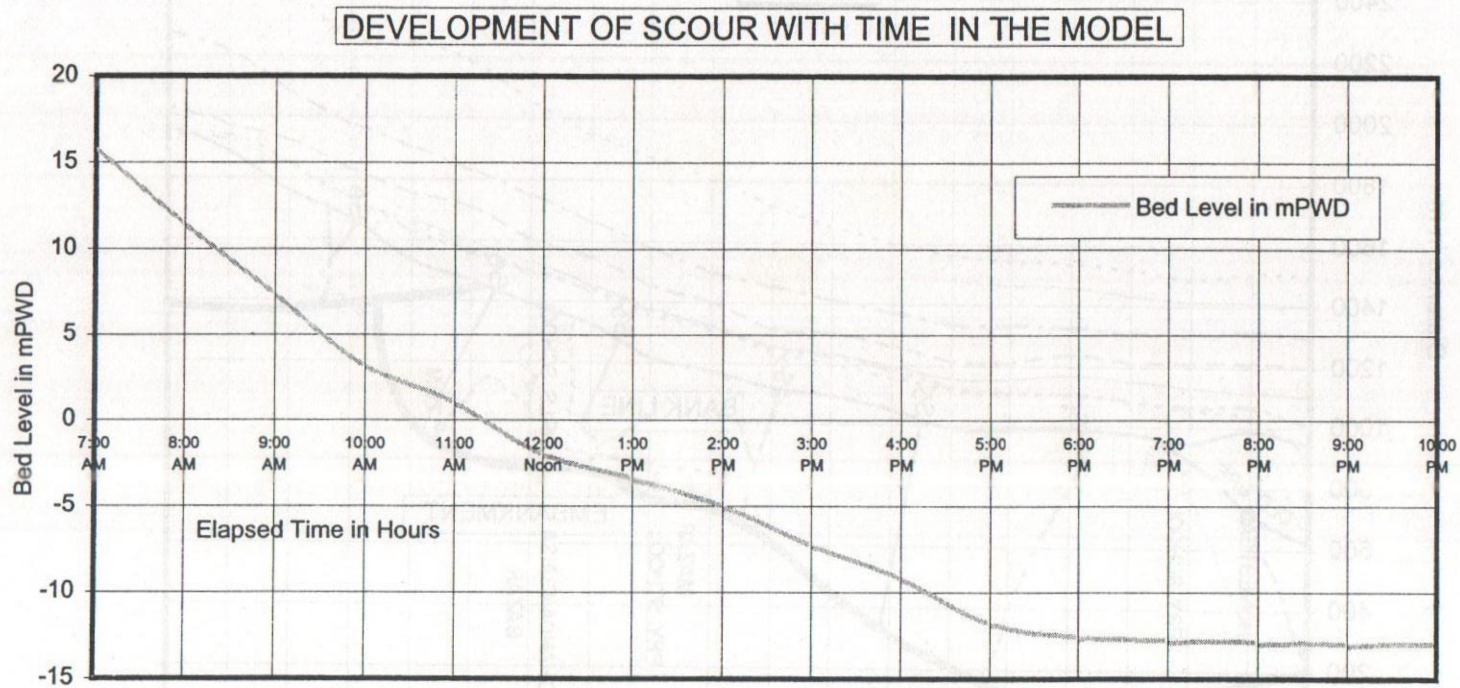


FIG-3: DEVELOPMENT OF SCOUR WITH TIME IN THE MODEL DOWNSTREAM OF SPUR NO.2 IN TEST NO.7

HYDRO-MORPHOLOGICAL PREDICTION AROUND THE EAST GUIDE BUND OF THE BANGABANDHU BRIDGE THROUGH A PHYSICAL MODEL STUDY

Syed Abdus Sobhan[1], Md. Abdus Samad[2], Md. Nazrul Islam Siddique[3] and Md. Abul Kashem[4]

Abstract

This paper presents the hydro-morphological behavior around the East Guide Bund (EGB) of Bangabandhu Bridge on the basis of physical model study. This study was a part of the Physical Model Study of the Jamuna River at the Bangabandhu Bridge area. An undistorted sectional model on EGB was setup to make the predictions with sufficient lead time. The sectional geometric and dynamic boundaries for the sectional model were selected from the results of the distorted overall fixed bed model studied with the same batheymetry. The study was focused mainly on the char erosion, deposition, flow & velocity pattern, scour pattern around EGB, and also, on the stability of the block placed on the slope & toe of EGB. Both qualitative and quantitative prediction based on the model results, are presented in this paper. In addition to these, scour pattern around the Bridge Piers are also discussed .

Introduction

The Bangabandhu Bridge Project is one of the largest projects ever implemented in Bangladesh. It includes a bridge having a curved length of 4.8 km and several components of river training works for guiding the flow to pass under the bridge span safely. The river training works comprised of East and West Guide Bunds having length of more than 2 km each. The East Guide Bund (EGB) was planned to be constructed on a flood plain About 250 m wide trench was dredged along the periphery of the guide bund for the ease of construction. The planform of the Jamuna River is characterized by a braided river alignment with a braided channel(typically 2 to 4 channel) and char pattern. The discharge of the Jamuna river varies roughly between 4000 m^3/s in the lean season and some 100,000 m^3/s in the monsoon, whereby about 48,000 m^3/s corresponds to bankfull discharge. The average water surface slope of the river varies within 5-9cm per km(Rendel et al, 1996). The river flows through fluvial depositions of fine sand and traces of silt.

The morphology of the Jamuna River is very lively. The river changes its course both laterally and cross-sectionally and thereby planform is changing every year very rapidly. Forecasting of probable morphological changes which may occur near the guide bunds is of immense importance for future maintenance of the river training works. However, forecast is necessary against the critical hydro- morphological changes around EGB. The sectional models are important to study in details of the various aspects because of planform changes with time. About 5 km of the river reach covering the river training works with partial width of the river was reproduced based on the float trackings performed in the overall fixed bed model.

[1]Director General, RRI, Faridpur; [2]Director (In Charge), Hydraulic Research, RRI, Faridpur
[3]Principal Scientific Officer (In Charge), RRI, Faridpur; [4]Scientific Officer, RRI, Faridpur

Objectives of model study

The specific objectives of the study are:

- To investigate the local scour around the EGB and bridge piers
- To check the flow pattern and velocity around the EGB
- To forecast possible morphological changes around the EGB
- To verify the stability of the stone/boulders against design discharge

Methodology of the study

In the scale model study, extreme flow velocities were investigated with the design discharge (91000 m^3/s) of the bridge. Only a part of the river covering EGB, work harbour and Dhaleswari spill channel were reproduced in this study (Fig.1). Discharge of the sectional width of the river was calculated as11000 m^3/s by area-velocity method. The lateral limit was derived from the flowlines observed in the Jamuna overall fixed bed model for the design discharge of 91000 m^3/s for the bathymetry data of August and September, 1996.. The size of the bed material in the model is not possible to scale down and the size of the bed material used in the model is almost similar to that of the prototype. To overcome this constraint and to simulate scour processes, by adapting the procedure of FAP 21/21, 1993, the discharge was increased in such a way so that the average velocity in the model was more than the critical velocity for sand movement in the model and the model was run until equilibrium scour reached.

Basic data

To conduct this physical model study, bathymetric data were collected from SWMC for the period of August-September 1996 and also from CSC of the bridge for the period of October 1996 & February, 1997. Particulars of EGB, Bridge and Bridge piers, etc. were collected from CSC. Hydrometric data such water levels, velocity, etc. were collected from the overall fixed bed model being considered as parent model. Bed material and bank material were also collected by SWMC from different locations of the surveyed reach of the river. These samples were analyzed at RRI.

Model design and construction

The model was designed according to the Froude's model law. Having considered the physical facilities of RRI, an undistorted scale of 1:90 was selected to ensure sufficient depth for precise measurement and to fulfill rough turbulent flow in the model. To ensure sediment movement in the model, discharge was increased for scour simulation in such way that average velocity was about 1.5 times greater than the critical velocity for sand movement in the model. The model design parameters along with scale factor are given in Table 1.

Table-1: Model design parameters and scale ratios

Parameters	Unit	Proto type	Model	Scale factor
Length	m	5100	56.7	90
Sectional width	m	1800	20	90
Cross- sectional Area	m^2	8130	1.004	8100
Design water level	+m PWD	14.55	14.55	-
Mean depth	m	7.88	0.088	90
Design discharge	m^3/s	11000	0.143	76843
Flow velocity	m/s	1.35	0.142	9.49

(Source : RRI-BUET, 1997)

About 5.1 km x 1.8 km river reach covering the total length (2.08km) of EGB, and Dhaleswari spill channel was reproduced in the model. The layout plan of the sectional model is shown in Figure 1 along with cross-section and water circulation system. To control and maintain required inflow, two rectangular sharp crested weir was installed at the upstream end of the model. Two well gauges were firmly installed to register the water levels in the model. A stilling basin was constructed at the upstream of the inflow section and hollow bricks were placed for energy dissipation to ensure uniform flow in the model. Eight tail gates were installed at the downstream end of the model to maintain required water levels.

Boundary conditions

In the study, the upstream boundary condition was the discharge distribution of 11000 m^3/s at the upstream limit of C/S-41 (2.10 km u/s of the Bridge axis). The downstream boundary condition in the model was the water level as +14.55 mPWD at C/S-76 (1.4 km d/s of the Bridge axis). The lateral boundary was a fixed wall which was constructed according to the flow line measured in the over all fixed bed model for the same batheymetry (Fig.1).

Measurements

Discharge was determined from the depth of water over the weir crest measured at upstream of two standard rectangular sharp crested weirs installed at the upstream end of the model. The water depth (h_w) was standardized for the direct measurements of the discharges according to the Rehbock's formula[Fap-21/22, 1993] and it is:

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

where, g= acceleration due to gravity (m/s^2); h_w= water depth over the weir crest (m); p= height of the weir crest above the bottom (m); Q= Discharge passing the weir (m^3/s); U_w= average flow velocity in the cross section in which h_w is measured (m/s); b= width of the weir.

The water levels were measured from the gauge readings throughout the model bed.Time averaged flow velocities were measured at 0.2 and 0.8 depth along the cross sections at a regular interval. The surface velocity at shallow depths were measured by floats and to obtain the mean surface velocity is multiplied by a factor of 0.85 (RRI-BUET, 1996). The slope of water surface in the model is determined from water levels registered at upstream and downstream gauges. Float tracking was performed at different cross sections in the model for determining flow lines. In the model, the scour depths were measured by inserting a meter scale into the scour hole with respect to a fixed reference line whose R.L. was known.

Model calibration

The calibration was done on the basis of the data collected from the overall fixed bed model which was represented as the prototype. The calibration results are presented in Table 2. Several calibration tests were performed to attain optimum prototype similarity.

Table-2: Comparison of parameters between prototype and model

Parameter	Unit	Prototype Data	Observe d value in model	Scale factor	Corresponding Prototype value in prototype
Discharge	m^3/s	11000	0.148	76843	11100
Mean depth	m	7.88	0.0878	90	7.90
Cross sectional area	m^2	8130	1.02	8100	8283.5
Ave. flow velocity	m/s	1.35	0.141	9.49	1.34
Water surface slope	-	$7.5 x 10^{-05}$	$5.4 x 10^{-}$	1	$5.4 x 10^{-04}$
Chezy's Co-efficient	$m^{0.5}/s$	92	21*	1	21

*Froudian similitude Chezy's Co-eff. in model(C)= 55 $m^{0.5}/s$ (Source : RRI-BUET, 1997)

Test runs

Two tests were performed with different test conditions to study the flow fields, stability of stones/ boulders and to quantify local scour around bridge and bridge piers.

Analysis and discussion

In different test scenarios, flow velocity, float tracking, and scour levels were recorded. Hence, flow velocity and equilibrium scoured bed levels found in the model are presented in the Table 3, 4, & 5. The pattern of flow lines and scoured bed levels observed in the model are shown Figure 2 & 3.

Table-3: Maximum point velocities at different cross sections ($Q=11000 m^3/s$)

C/S no	Location			Maximum velocity m/s
	Distance of C/S from EEP(m)	Distance from periphery of EGB(m)	Distance from left boundary(m)	
45	1700 u/s		1530	3.14
50*	1200 u/s	315		3.24
53*	900 u/s	1350		3.14
59*	300 u/s	90		2.75
60*	200 u/s	90		2.46
65*	300 d/s	135		2.39
69*	700 d/s	270		1.71
72	1000 d/s		675	1.20
73	1100 d/s		450	1.20
76	1400 d/s		495	2.39
80	1800 d/s		990	2.49
85	2300 d/s		1485	2.68

* C/S around EGB (Source : RRI-BUET, 1997)

From the observation of flow lines, it was found that maximum flow concentration was towards the EGB (Fig.2). As a result, there was a tendency of developing a channel to link the dredged trench around EGB with the main river from upstream direction. Flow through this channel might attack the slope of EGB at about 800 m upstream of the bridge. From the Table 4. it was observed that the stone/boulders were found stable against a velocity of 3.24 m/s.

From the scour test, erosion was observed along the outer periphery of the char infront of the EGB at the downstream of the bridge and some deposition took place in the work harbour and also near the Dhaleswari offtake. On the other hand, there was a tendency of erosion of the adjacent large sand bars at immediate down stream of the work harbor. From the Table 4 and Figure 3, it was found that the lowest equilibrium scoured bed levels around the toe of EGB did not exceed the design level of either -15.0 mPWD or -18.0 mPWD.The lowest equilibrium scoured bed level around bridge pier was found as -1.30 mPWD at the pier no.46 which is located at a distance of about 270m from the periphery of the EGB(Table 5).

Table-4: Equilibrium scoured bed level at different sections

C/S no	Location			Scoured bed level (mPWD)
	Distance of C/S from EEP (m)	Distance from Periphery of EGB (m)	Distance from left boundary[480500] (m)	
44	1800 u/s		630	+8.01
50*	1200 u/s	135		-7.20
52*	1000 u/s	135		-5.13
53*	900 u/s	135		-3.33
55*	700 u/s	135		-5.67
60*	200 u/s	135		-5.49
65*	300 d/s	135		-8.64
69*	700 d/s	135		-9.09
72	1000 d/s		45	-0.09
76	1400 d/s		675	+7.2
78	1600 d/s		630	+6.66
80	1800 u/s		360	+7.02

Table- 5 : Equilibrium scoured bed levels around bridge piers for Test 4

Pier No. [from EGB]	Initial bed level at pier (mPWD)	Scoured bed elevation at pier (mPWD)	Scour depth (m)
46	+10.71	-1.30	12.01
45	+11.61	+8.6	3.01
44	+10.08	+9.50	0.58
43	+12.60	+9.50	3.10
42	+12.51	+8.60	3.91
41	+12.60	+9.50	3.10
40	+12.24	+8.60	3.64
39	+11.43	+7.70	3.73
38	+11.40	+9.50	1.90
37	+10.70	+7.70	3.00
36	+9.20	+8.60	0.60
35	+8.10	+6.80	1.30
34	+6.02	+5.00	1.02

(Source : RRI-BUET, 1997)

Conclusions

The scale model study of the EGB was conducted on the basis of Bathymetry August-September October,1996 and Febrauray1997 to predict the possible hydro-morphological changes around EGB. The detail analysis of the results and the subsequent discussion on the basis of the physical model test are the basis for the conclusion given here. A channel may be developed to link the dredged trench around the guide bund with the main river from upstream direction and flow through this channel will be concentrated in the vicinity of the slope of EGB. The stone/boulders may be stable against the velocity of 3.24 m/s. Char infront of EGB will be eroded incourse of time. Deposition will take place in work harbor and near Dhaleswari offtake. Large sand bar infront of Dhaleswari offtake will be eroded gradually. Deposition will take place on the toe of EGB. The maximum scour may be occured at pier no. 46 about 270 m from the periphery of EGB.

Acknowledgment

The authors would like to acknowledge all officers and staff of RRI helped in carrying out the study works.

Abbreviation

CSC	Construction Supervision Consultants (Rendel Palmer & Tritton/Nedeco/BCL)
EGB	East Guide Bund
EEP	East end pier
JMBA	Jamuna Multipurpose Bridge Authority
RRI	River Research Institute
SWMC	Surface Water Modelling Centre

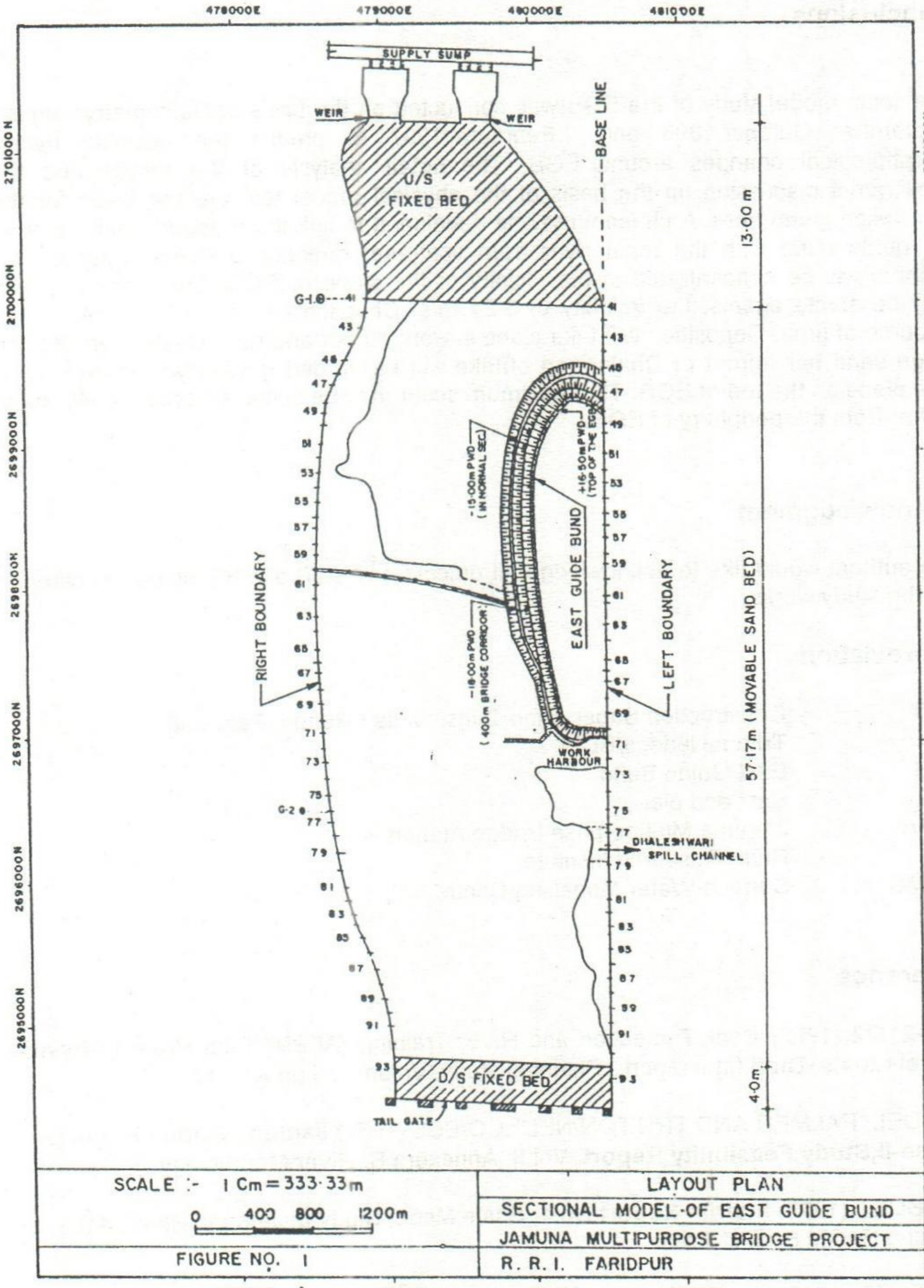
Reference

FAP-21/22 (1993) Bank Protection and River Training (AFPM) Pilot Project,**Physical models tests**, Draft final report, Planning study, Volume VI,pp A14-12.

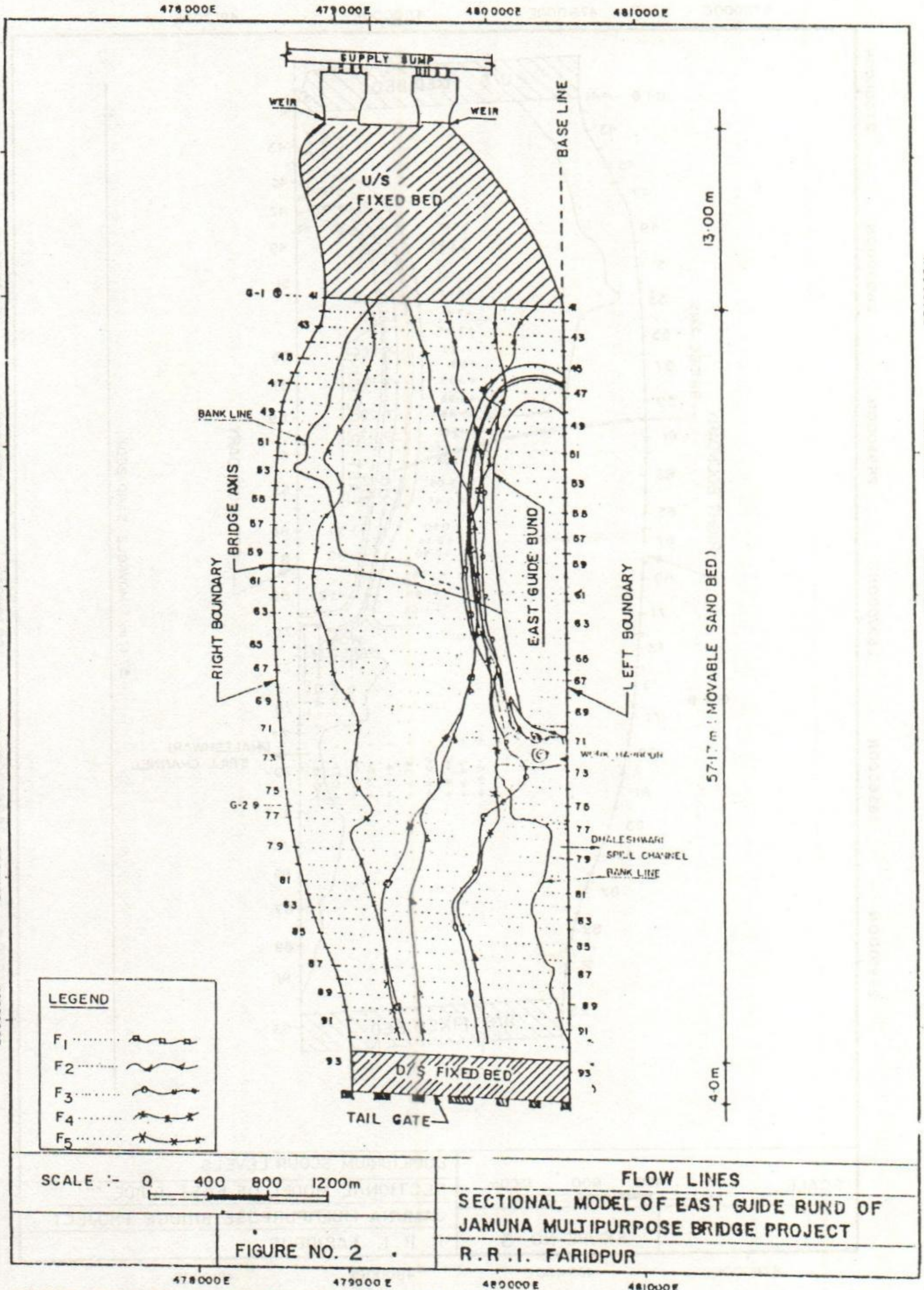
RENDEL, PALMER AND TRITTON/NEDECO/BCL (1989) **Jamuna Bridge Project, Phase-II,Study Feasibility Report**, Vol II, Annexure B: River Morphology.

RRI-BUET (1996) **Interim Report No.1**, Scale Model Study of Jamuna River at the Jamuna Bridge Project Area.

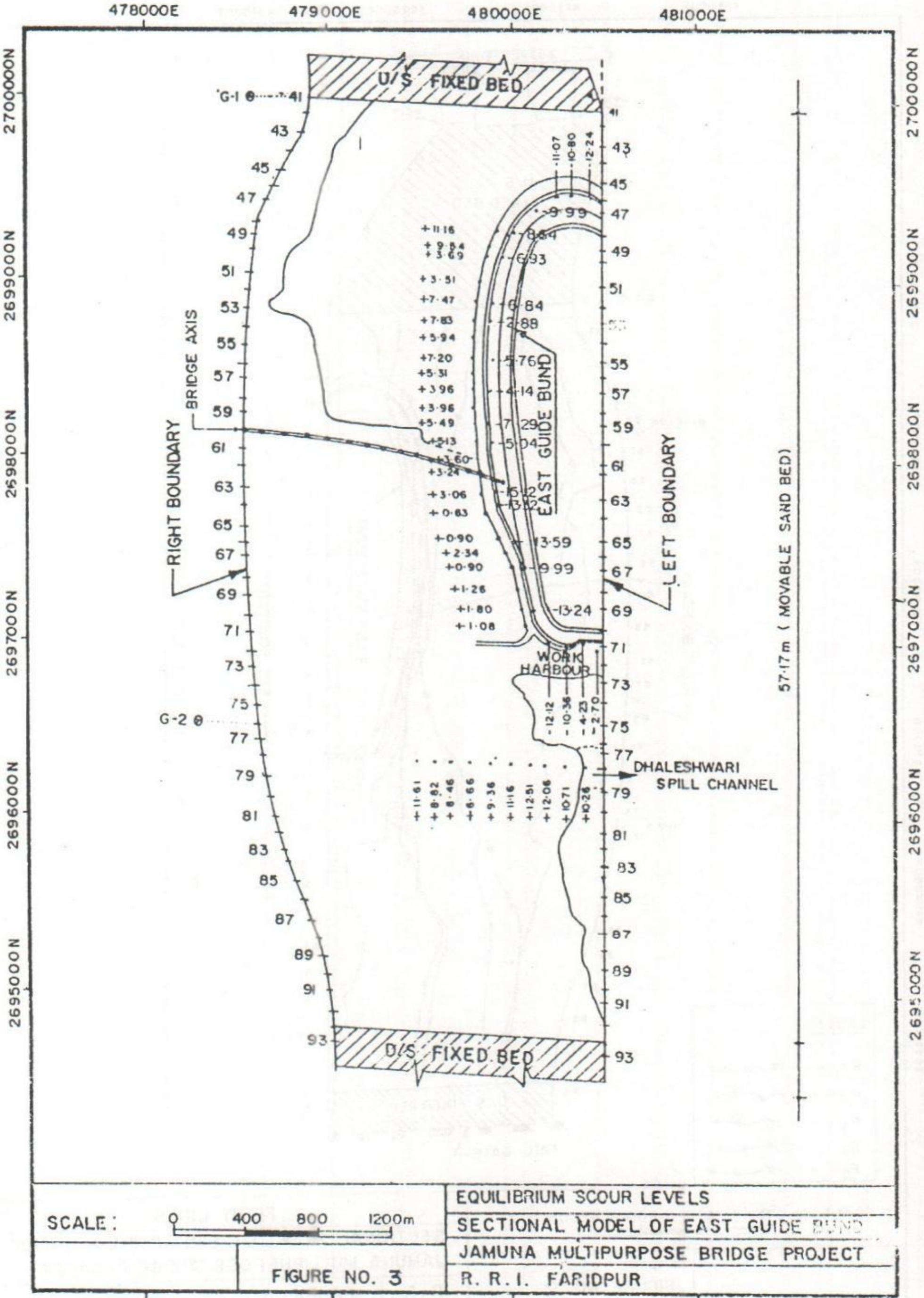
RRI-BUET (1997) **Interim Report No. 2**, Scale Model Study of Jamuna River at the Jamuna Bridge Project Area.



SOURCE : RRI-BUET, INTERIM REPORT NO. 2, AUGUST, 1997



SOURCE : RRI-BUET, INTERIM REPORT NO. 2, AUGUST, 1997



SOURCE : R R I-BUET, INTERIM REPORT NO. 2, AUGUST, 1997

CONSOLIDATION CHARACTERISTICS OF SOIL OF DHAKA INTEGRATED FLOOD PROTECTION PROJECT AND RELATIONSHIPS BETWEEN COMPRESSION INDEX AND SOME OTHER PARAMETERS

Mst. Anwara Jahan[1], A M M Motaher Ahmed[1], Md. Hanif Mazumder[2], Md. Rafiqul Alam[3] and S H M Shamim Kabir[4]

Abstract

The consolidation characteristics of foundation soil is mainly characterized by primary consolidation and secondary consolidation. In this paper consolidation characteristics of the soils of the Dhaka Integrated Flood Protection Project area have been discussed and some relationships between compression index and other index properties have been established for a simple scenario based on statistical analysis. A good correlation between compression index and initial void ratio is found in the present investigation.

Introduction

Bangladesh experienced two successive severe catastrophic flood events in 1987 and 1988. In 1988, major areas including the capital city of Bangladesh was inundated. FAP 8B (1991) reported that 77% (i.e. 200 km^2)of the city was flooded to an incomparable degree with flood depths ranging from 0.3 m to 4.5 m and in some cases more than 4.5 m and about 60% of the total population was directly affected by these floods.

Giving emphasis to these devastating floods, the Government of Bangladesh has undertaken a project to take preventive measures against Floods and Drainage problems within the Greater Dhaka city. Initially, an area of about 137 km^2 in the highly populated western part of the city was planned to be protected including construction of about 30 km of embankment and 7 km of flood protection wall along the westerln periphery of the city, complemented by about 2 km of new roads, 8.5 km of road raising to the east, pipe sluices, cleaning and repair of internal drainage khals and sewerage systems. Additional works were taken up at the same time included construction of a flood protection bund around Zia International Airport and 30 km of flood protection wall around the Dhaka-Narayangonj-Demra Zone to the south of the city (Draft Report on FAP, 1994).

The overall flood protection program for Dhaka involves remedial works and completion of the western embankment (30 km), internal embankments, construction of the eastern embankment (29 km), road raising, construction of additional sluices and pump stations, and improvement of storm water drainage.

---

[1]Scientific Officer, RRI, Faridpur; [2]Director (In Charge), Geotechnical Research, RRI, Faridpur
[3]Principal Scientific Officer (In Charge), RRI, Faridpur; [4]Senior Scientific Officer, RRI, Faridpur

Soil samples at different locations within the project area were collected and laboratory investigation was performed at RRI to evaluate the soil properties of the embankment and subgrade soils which affect performance and design. Different types of analysis were performed such as Atterberg Limit Tests, Sieve Analysis, Hydrometer tests, one-dimensional Consolidation tests, Triaxial Consolidation Tests etc. The results of the tests are available in RRI Soil Testing Reports (RRI Reports, 1992).

When a saturated clay-water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores resulting hydraulic gradients initiate a flow of water out of the clay mass and the mass begins to compress. A portion of the applied stress is transferred to the soil skeleton, which in turn causes a reduction in the pore pressure. This process, involving a gradual compression occurring simultaneously with a flow of water out of the mass and with a gradual transfer of the applied pressure from the pore water to mineral skeleton is called consolidation.

Here, the aspects of primary and secondary consolidations and their related parameters are discussed. Primary consolidation results from reduction in volume due to outflow of water and air from soil pores. Secondary consolidation results due to plastic deformation of soil mass under constant load. This paper aiming at evaluation of consolidation characteristics by only compression index and its relationship with other important soil parameters. In addition, some theoretical consequences and related properties are described in short to have an understanding about their implications with compression index.

Soil characteristics of Dhaka city

Dhaka city is located on the southern part of the Raised Alluvial Terrace, the Modhupur Tract. The Dhaka clay is known to be over-consolidated. Undrained shear strength, S_u, of the Dhaka clay (LL= 41-43%, PI =22-23%) (Shafiullah, 1994).

FAP 8B reported that due to the very low shear strength of the subsoil within the project area, sudden failure of the embankment was occurred in some places. The damage could have been reduced by proper compaction and moisture control during construction. The western flood protection embankment was subjected to damage and the reasons were identified as lack of moisture control, inadequate compaction, inadequate subgrade improvement and inadequate wave protection. In many locations, the subgrade shear strength was inadequate to support the load of the embankment (FAP 8B, 1991).

Dhaka clay is subjected to be isotropic and one dimensional (k_0) consolidation. Undrained strength test results from their normally consolidated samples showed that the Dhaka clay has S_u/σ_v values of 0.30 and 0.19 for isotropic and k_0-consolidated conditions respectively. With plasticity index of 22 to 23 percent of the soil it is seen that results for k_0 consolidated samples are very close to the Skempton's equation (Shafiullah, 1994).

Review of literatures

The magnitude and rate of settlement of silt-clay soils caused by consolidation under foundation load are measured by consolidation test. For this purpose it is necessary to obtain undisturbed soil samples from various depths and various points of the site (Murthy, 1993).

The total compression of a saturated clay strata under excess effective pressure may be considered as the sum of immediate compression, primary compression and secondary compression. The value of the co-efficient of secondary consolidation, C_α is less for sandy silt and stiff clays and relatively high for organic and soft clays (Islam & Roy, 1991).

Settlement behaviour of relatively low permeable soils can be predicted from the results of consolidation test (Alimuddin, 1991).

The strength of the soil generally decreased with an increase in amount of water in the soil pore and in the pressure existing in this pore water. Generally, high compressibility is associated with low shear strength, and as the degree of consolidation can not be reliable determined in advance, the shearing resistance at any given time is also a matter of considerable uncertainty (Murthy, 1993).

Some basic equations for evaluation of consolidation

Many equations were developed by many investigators which are available in text books. Of them, related equations are presented here to envisage the parameters responsible for primary or secondary consolidation.

The co-efficient of secondary consolidation, C_α can be calculated from the following equation :

$$C_\alpha = \frac{\Delta e}{1+e_o} \frac{1}{Log_{10} \frac{t_2}{t_1}} = \frac{C_t}{1+e_0} \tag{1}$$

where, $C_t = \dfrac{\Delta e}{Log_{10} \frac{t_2}{t_1}}$

Where, C_t = Secondary compression index, e_0 = initial void ratio, Δe = change in void ratio, t_1 = initial time and t_2 = final time

The slope of the straight line portion of the e-logt curve is known as the secondary compression index, C_t. Numerically, $C_t = \Delta e$ for a single cycle of time on the curve. The additional settlement, S due to the secondary compression can be estimated as S $=2HC_\alpha$, where 2H represents the total thickness of the soil layer for two phase drainage.

Time of settlement can be determined from the equation as follows:

$$t = \frac{T_V}{C_V H^2} \qquad (2)$$

Where,

t = time required to reach a certain percentage of primary consolidation
T_v = Time factor co-efficient dependent on percentage of consolidation
C_v = Volume of compression and H= thickness of layer

The Compression Index (C_c), Co-efficient of Compressibility (a_v) and Co-efficient of Consolidation (C_v) are soil parameters which are used in computation of settlement characteristics of sub-surface soils under structural load and these parameters are usually determined from the conventional laboratory consolidation tests. From laboratory consolidation test the settlement, S can be computed from the following equation:

$$S = H\frac{C_c}{1+e_0} Log_{10}\frac{P_0+\Delta P}{P_0} \qquad (3)$$

Where H is the thickness of the bed of the soil under pressure, P_0 and ΔP is the pressure difference from P_0 to $P_0+\Delta P$ due to structural load.

The compression index, C_c is equal to the slope of the linear portion of the void ratio versus Logσ curve,

$$C_C = \frac{-\Delta e}{Log\frac{\overline{P}}{\overline{P_0}}} = -\frac{\Delta e}{Log\frac{\overline{\sigma}}{\overline{\sigma}_0}} \qquad (4)$$

Where, $\overline{\sigma}_0$ = initial effective stress
$\overline{\sigma}$ = final effective stress
Δe = change in void ratio

When difference in void ratio corresponding to one log cycle, the equation (4) becomes,

$C_c = -\Delta e/1 = -\Delta e$

The compression index, C_c is also related to initial void ratio (e_0), liquid limit (LL) and natural water content (NWC). Using the basic equations, many empirical equations were developed for different conditions of soil.

Terzaghi and Peck (1967) found empirical relationships with compression index and liquid limit, initial void ratio and natural water content separately for undisturbed clay soil as follows:

$C_c = 0.009\,(LL - 10)$ (5)

$C_c = 0.54\,(e_0 - 0.35)$ (6)

$C_c = 0.0054(2.6\,\omega_0 - 35)$ (7)

Where, LL= liquid limit in percent, e_0 = initial void ratio and ω_0 = water content

The co-efficient of compressibility a_v may be calculated from the compression index as,

$a_v = 0.435\,(C_c/\sigma_a)$ (8)

Where σ_a is the average pressure for the increment.

Historically, in soil Mechanics, Skempton's (1957) equation has been widely used for normally consolidated clays as follows:

$S_u/\sigma_{v0} = 0.11 + 0.0037\,PI$ (9)

Where S_u/σ_{v0} is the Undrained Strength ratio, S_u is the Undrained Shear Strength of soil, σ_{v0} is the effective overburden pressure and PI is the Plasticity index.

Methodology

Laboratory consolidation tests were performed on various undisturbed soil samples. Among them, only the test results of 53 samples within the project boundary were taken for analysis. The values of NWC, LL, PI, density for individual samples were measured and Initial Void Ratio and C_c values were calculated separately. The samples were collected by 3-4 inch Shelby tubes. The samples were carefully trimmed to fit into the Consolidometer ring. Various load increments were applied and allowed to remain in the machine for 24 hours. For each pressure increment, compression readings were taken at required intervals.

Type of the materials

Visual inspection and tests revealed that all most all the samples were clay with trace fine to medium sand. A very few samples contained organic matters and a very little amount of silts were traced. The properties of the soils based on test results are elaborated in Table 1.

Analysis and discussion

The values of Natural Water Content (NWC), Liquid Limits (LL) and Plasticity Index (PI) are plotted individually against C_c values and the established relationships are shown in Figures 1 to 3. The test results revealed that compression index, C_c ranges from 0.132 to 0.67 excluding one extreme value of 1.55. The plasticity index ranges from 14 to 44 excluding one extreme value of 69. The liquid limit varies between 35 to 86 with one extreme value of 131. Natural Water Content lies within 19.33% to 84.01% with one extreme value of 127.69. The extreme values indicates that the samples are organic clay.

From the analysis of data, the following regression equations are established :

$$C_c = 0.0073\ (LL - 14.57) \qquad (10)$$

$$C_c = 0.4319\ (e_0 - 0.286) \qquad (11)$$

$$C_c = 0.0054(1.78\ \omega_0 - 10) \qquad (12)$$

The graphs show that C_c increases with the increase of values of LL, ω_0 and e_0. The equations show conformity with the equations developed by Terzaghi and Peck (1967).

The regression co-efficient for C_c versus e_0 curve is found 0.96 which indicates that the initial void ratio is better correlated with the compression index. The regression co-efficient for natural moisture content is found to be slightly less than initial void ratio and on the other hand liquid limit is poorly correlated with compression index.

Conclusions

The compression index is useful for estimation of the settlement in the field as well as in the laboratory also. The Co-efficient of secondary compression, C_α depends on the void ratio, liquid limit, compression index but not depends on the increment of pressure. The compression index indicates the type of soil and the settlement characteristics of the foundation soil and also the maximum load bearing capacity of the soil can be envisaged knowing the values of compression index. The established relationships so far been developed may give clear insight to the designers for designing structures within the project area. The investigation shows a good correlation between compression index and initial void ratio. The other relationships have some limitations for use.

References

Alimuddin, A. (1991) **Effect of particle sizes of clay and silt on the compressibility and consolidation characteristics**, Technical Journal, RRI, pp 43-53.

Dhaka Integrated Flood Protection Project, FAP 8B (1991) **Final Report**, Louis Berger Int'l Ltd., FPCO/GoB, Dhaka.

Islam, A,M.Z. & Roy, P.K. (1991) **Secondary Compression Characteristics of Clays for Some Areas of Bangladesh**, Technical Journal, RRI, pp 35-42.

Murthy, V.N.S. (1993) **Soil Mechanics and Foundation Engineering in SI Units**, Fourth edition, New Delhi, India.

Shafiullah, A.M.M. (1994) **Some Geotechnical aspects of Bangladeshi soils**, Paper Presented at IEB, Dhaka, Bangladesh.

Skempton, A.W. (1957) **Discussion on the Planning and Design of the New Hong Kong Airport**, Proc. of ICE, Vol. 7, pp 306.

RRI Reports [Report No. 24(92), 39(92), 46(92), 55(92), 5(61), 62(92), 68(92), 69(92), 73(92), 83(92), 91(92)], RRI, Faridpur.

Terzaghi, K. and Peck, R. B. (1967) **Soil Mechanics in Engineering Practice**, 2nd edition, John Wiley & Sons Inc., New York.

Table 1: Summary of Test Results

Location : Segunbagicha, Maniknagar

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3) Dry	Density(KN/m^3) Wet	e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
1	38.56	53	26	19.05	13.75	0.7538	2.676	0.169	10.112	Stiff CLAY
2	24.9			18.37	14.72	0.8755	2.664	0.249	10.112	Sandy SILT
3	24.54	47	23	18.28	14.63	0.8929	2.678	0.239	10.112	Sandy CLAY
4	20.52	44	20	19.18	15.91	0.8152	2.676	0.235	10.112	Stiff CLAY with fine sand
5	41.82	46	21	17.25	12.27	1.264	2.682	0.362	10.112	Stiff CLAY with fine sand
6	22.25	42	18	19.35	15.83	0.7915	2.675	0.242	10.112	Stiff CLAY with fine sand
7	127.69	52	26	14.01	6.15	3.414	2.625	1.55	4.08	Organic CLAY
8	19.33	46	21	19.84	16.55	0.5464	2.676	0.135	8.16	CLAY

Location.: Rajabazar, Katalbagan, Begunbari & Ibrahimpur

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3) Dry	Density(KN/m^3) Wet	e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
1	24.22	62	35	19.89	16.01	0.6513	2.683	0.22	10.112	Sandy CLAY
2	27.3	67	39	18.84	14.8	0.8288	2.685	0.228	10.112	Sandy CLAY
3	24.66	39	19	19.8	15.89					Sandy CLAY
4	41.88	59	33	18.67	13.16	1.2311	2.682	0.397	10.112	Sandy CLAY
5	34.99	67	40	17.43	12.91	0.9952	2.686	0.242	10.112	Sandy CLAY
6	24.6	55	29	19.4	15.57	0.8553	2.68	0.24	10.112	Sandy CLAY

Table 1 : Continued

Location : Segunbagicha khal

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3)		e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
				Dry	Wet					
1	53.375	54	27	17.03	11.1	0.994		0.276	8	CLAY
2	37.69	67	37	18.22	13.24	1.0462		0.408	8	Sandy CLAY
3	25.16	57	30	18.62	14.88	0.7262		0.182	8	Sandy CLAY
4	42.67	70	40	17.28	12.11	0.9802		0.286	8	Sandy CLAY
5	58.1	60	30	15.73	9.95	1.5445	2.67	0.52	10.112	Organic CLAY
6	29.95	57	29	19.75	15.19	0.8776	2.682	0.205	8	Sandy Stiff CLAY
7	38.13	65	37	17.65	12.78	1.8137	2.665	0.665	10.112	Sandy CLAY
8	84.01	86	42	15.11	8.21	1.8137	2.665	0.665	10.112	Organic CLAY

Location : Mohakhali Khal

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3)		e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
				Dry	Wet					
1	92.98	131	69	15.56	8.06	1.5893	2.665	0.463	10.112	Organic CLAY
2	51.29	54	27	18.32	12.11	1.2543	2.676	0.425	10.112	Sandy organic CLAY
3	58.08	60	32	16.41	10.65	1.43	2.67	0.35	8.16	Sandy Organic CLAY
4	45.88	58	30	16.77	11.5	0.53	2.682	0.306	8.16	Sandy CLAY

Table 1 : Continued

Location :Segunbagicha

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3) Dry	Density(KN/m^3) Wet	e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
1	31.52	58	33	19.39	14.75	0.8748	2.685	0.247	10.112	Sandy CLAY
2	22.43	44	20	18.26	14.91	0.7702	2.672	0.203	10.112	Sandy CLAY
3	42.33	62	33	17.79	12.5	1.206	2.669	0.378	10.112	Organic CLAY
4	44.77	60	30	18.21	12.58	1.265	2.665	0.34	8	Sandy Organic CLAY
5	36.23	59	32	17.16	12.59	1.0492	2.683	0.352	8	Sandy CLAY

Location : Tejkunipara

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3) Dry	Density(KN/m^3) Wet	e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
1	26.01	58	30	18.53	14.71	0.7705	2.683	0.207	10.112	Sandy CLAY
2	23.37	57	29	19.98	16.2	0.6477	2.682	0.132	10.112	Sandy CLAY

Table 1 : Continued

Location : Begunbari Khal

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3)		e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
				Dry	Wet					
1	62.65	75	35	16.11	9.9	1.764	2.665	0.67	10.112	Organic CLAY
2	27.26	35	14	18.95	14.89	0.8	2.674	0.254	10.112	CLAY
3	24.81	41	17	18.19	14.57	0.721	2.675	0.188	10.112	Sandy CLAY

Location : Uttara

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3)		e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
				Dry	Wet					
1	64.21	76	39	15.89	9.85	2.0699	2.665	0.67	8.18	Organic CLAY
2	45.19	58	28	16.6	11.43	1.2575	2.678	0.475	8.18	Organic CLAY
3	34.62	39	15	18.76	13.93	0.881	2.675	0.179	8.16	Organic CLAY with sand
4	22.96	38	15	19.44	15.8	0.7426	2.68	0.193	8.16	Sandy CLAY
5	36.4	45	19	18.5	13.53	0.9595	2.678	0.285	8.16	Sandy CLAY

Table 1 : Continued

Location : Begunbari and Mohakhali Khal

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3) Dry	Density(KN/m^3) Wet	e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
1	25.62	38	16	18.16	14.44	0.7803	2.68	0.225	10.112	Sandy CLAY
2	39.38	48	20	17.28	12.4	1.1937	2.676	0.345	8	Sandy CLAY
3	29.77	74	44	18.73	14.44	0.7325	2.68	0.1891	10.112	Sandy CLAY
4	39.29	55	27	18.32	13.16	0.7585	2.682	0.222	8	Sandy CLAY
5	47.94	51	25	16.64	11.25	1.129	2.673	0.4022	10.112	Sandy Organic CLAY
6	53.43	60	30	16.12	10.51	1.439	2.624	0.475	10.112	Organic CLAY
7	56.66	64	31	17.11	10.93	1.2686	2.671	0.367	10.112	Organic CLAY

Location : Unknown

Sl.No.	N.W.C %	LL %	PI %	Density(KN/m^3) Dry	Density(KN/m^3) Wet	e_0	Relative Density	C_c	Maximum Load Kg/cm^2	Soil type
1	32.15	62	35	19.87	15.04	0.7074	2.685	0.204	10.112	Sandy CLAY
2	25.08	56	28	20.17	16.12	0.6987	2.682	0.187	10.112	Sandy CLAY
3	36.01	55	29	18.77	13.8	0.4168	2.683	0.229	8	Sandy CLAY
4	30.35	57	33	19.45	14.92	0.7382	2.684	0.238	10.112	Sandy CLAY
5	27.75	49	25	19.19	15.02	0.7316	2.68	0.162	10.112	Sandy CLAY
6	23.29	55	29	19.08	15.47	0.7018	2.682	0.143	8.16	Sandy CLAY

Figure 1 : Relationship between Compression Index and Natural Water Content

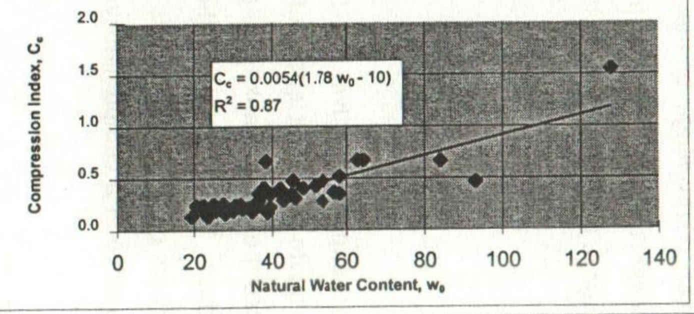


Figure 2 : Relationship between Compression Index and Liquid Limit

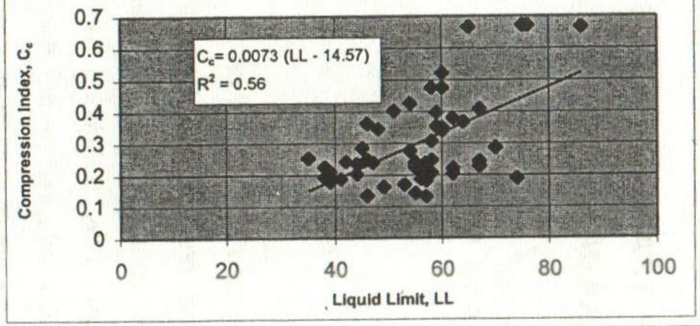
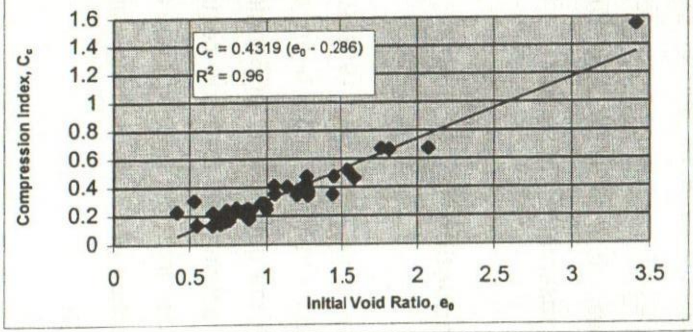


Figure 3 : Relationship between Compression Index and Initial Void Ratio



PERMEABLE GROYNES AS RIVER TRAINING WORKS : A CASE STUDY OF THE JAMUNA RIVER AT KAMARJANI

Md. Abdus Samad[1], Swapan Kumar Das[2], A.K.M Ashrafuzzaman[3],
Syed Md. Mymur Sultan[3] and Md. Manjurul Haque[3]

Abstract

Flood Action Plan component 21 (FAP 21) studied and worked on for specifying and constructing different types of groynes and revetments. Under these activities permeable groynes as a test structure was constructed on the Jamuna river at Kamarjani, in the district of Gaibanda in February 1994. But during the monsoon 1995, marked by five flood peaks, the structures were attacked by the flow causing some damages. As a result an adaptation physical model study was carried out to find out the causes of damages and to get more information/knowledge of the behavior of the groynes/groyne field in order to formulate design rules and to workout guidelines and manuals for their application. The overall findings of the adaptation physical model study are presented in this paper.

Introduction

In the year 1987 and 1988 Bangladesh has experienced two most severe floods. Several initiatives have been launched to mitigate the worst effect of such floods. In 1989, the government of Bangladesh requested the World Bank to co-ordinate the five year action plan for flood control in Bangladesh. The objective of the FAP is to co-ordinate individual approaches and major projects on water resources, drainage, flood protection and river training works in order to arive at a comprehensive plan for future design.

---

[1]Director (In Charge), Hydraulic Research,RRI, Faridpur; [2]Senir Scientific Officer, RRI, Faridpur
[3]Scientific Officer, RRI, Faridpur

Erosion of rivers banks is a common phenomenon of many rivers in Bangladesh. But, this situation may be worst in case of the Jamuna river. So the object of the Bank protection and River training (AFPM) pilot project (FAP-21) is to develop and optimize design criteria for bank protection as well as cost effective construction and maintenance methods which will be most appropriate for prevailing conditions and for future standards at the Jamuna and other rivers of Bangladesh. Such solutions could be found by designing and specifying different types of protective works at different safety levels, applying structures of different geometry using different materials and different protective layers and by investigating into suitable methods of construction. Under this pilot project a number of permeable groynes were constructed at Kamarjani area accross the Jamuna river as per recommendation from physical model studies and their performance and effectiveness in the field were monitored. On the other hand, river training/activated flood plain management component (FAP-22) aims to develop a measures which could allow to control the Jamuna river in such a way that its outer channels would no longer threaten their outer bank erosion. In addition, the width of the river would be reduced which will help to increase cultivable land. It was also tried to find out a solution for stabilizing the planform of the river by developing an alternative approach to apply "hard measures" for river training. "Soft" or recurrent measures were considered in order to "convincing" the river rather than forcing it to reduce its threatening and unpredictable behavior into a gentle and predictable one. Under FAP 21/22 activities construction of the groyne as test structure on the Jamuna river at Kamarjani was finalized in February' 1994. But during the monsoon' 1995 marked by five flood peaks, the structures were attacked by the flow causing some damages. This adaptation physical model investigations were carried out to find out the causes of damage of the groynes during the monitoring period. Moreover this study was also carried out to formulate design rules and to work out guide lines and manuals for the application of groynes.

Permeable groyne

Permeable groynes are pile structures, constructed transverse to the river flow and extended from the bank into the river. The term "permeable" means the groyne allows certain amount of flow to pass trough it. Permeable groynes simply obstruct the flow, reducing its velocity and causing silt deposition. They are, therefore, best suited for rivers carrying huge sediment load. In comparatively clear rivers they reduce the erosive strength of the current and thus prevent local bank erosion. Permeable groynes do not change the flow abruptly as is done by impermeable groynes, and hence, intense & serious eddies and scour holes are not developed. They are cheaper and perhaps the best for silt laden rivers. When the groyne is to be submerged, then permeable groynes give much better results, because they do not generate so strong turbulence as is generated by submerged-impermeable groynes, making them susceptible to be washed away due to over-topping (Garge, 1987).

Modeling area

Total 1.75 km of river reach was considered for model study at Kamarjani area including three permeable groynes G_2, G3, GA (Figure 1). The width of the modeled area was 350 m considering the existing facilities of RRI and the section width between the groynes head and the lateral left boundary was so selected that the influence of the groynes is as minimum as possible. The model area was designed to see the effect of angle of flow attack from 0^0 (parallel to the bank) to 45^0 to investigate the influence of parameters as the direction of flow attack changes.

Methodology

The scale of the model is determined to reproduce the area defined previously, taking into account the facility of the River Research Institute, Faridpur and to ensure that the scale laws were fulfilled. As a consequence, the scale 1 : 55 has been chosen. Critical velocity for sand movement in the model and prototype was calculated using the formula *(Van Rijn, 1984)*

$$V_{cr} = 0.19(D_{50})^{0.1} Log \frac{12h}{3D_{90}}$$

It was established that local scour depth is a function of sediment size and velocity of the flowing water if $V_{cr}<V<2.5V_{cr}$ (Ahmed, 1953). Flood Action Plan (FAP) studied that scour depth becomes independent of flow velocity if $V>2V_{cr}$ and thus maximum equilibrium scour will be reached earlier (FAP-21/22, 1993).

The characteristic values of the main parameters of this physical model are presented as follows:

Scale		1:1	1:55		
Model area length	m	1450	26.4	Water level (PWD)	+22.
Model area width	m	850	15.5	Mean bed level (PWD)	11
Width of the river represented	m	350	6.4	(u/s section)	
Water depth	m	11.4	0.21	Water depth (m)	11.4
Scour depth	m	15	0.27		
Velocity (Froude condition)	m/s	3	0.40	Bed level in the bend	+5
Discharge (Froude condition)	l/s	11949	0.533	Scour hole elevation	-10
CC Cube (falling apron)	m	0.3	0.005	Scour depth (m)	15
CC Cube (falling apron)	m	0.2	0.004		
CC Cube (falling apron)	m	0.1	0.002	V > 2 * Vc (Scour condition)	

Boundary conditions

In this adaptation physical model study three types of boundary conditions are considered: (i) an upstream boundary condition at the inflow of the model (ii) a downstream boundary condition at the outflow of the model and (iii) a lateral boundary condition, because in the model the whole width of an outflanking channel have not been modeled. The upstream boundary condition is the discharge and the discharge distribution along the upstream boundary of the model. The discharge of the model is chosen in order to get a mean prototype flow velocity in the upstream section of 3 m/s, which is the velocity observed during the flood events of 1995. The downstream boundary condition is the water level adjusted at the downstream of the model by operating tailgates. This water level is corresponding to the water level recorded in 1995. The model does not represent the total width of the outflanking channel, but only a part of about 350 m from the right bank. The lateral boundary condition is the location of the left boundary of the model. This model boundary is not the left bank of the outflanking channel, but it is a flow line of the prototype in order to represent a portion where the discharge is constant. The width of the model is sufficient to prevent influence by the groynes on this flow line. This boundary flow line was determined with the measurements of float tracking performed during the former model studies (FAP 21/22, 1996).

The required maximum discharge necessary in the model is around 540 l/s. This discharge in the model was measured by a sharp crested weir installed at upstream of the model using the following Rehbok's formula:

$$Q = \left(0.403 + 0.053 * \frac{h_w}{p}\right) \sqrt{2g} * b \left[(h_w + \frac{u_w^2}{2g})^{1.5} - (\frac{u_w^2}{2g})^{1.5} \right]$$

in which

b	=	width of the weir
g	=	acceleration due to gravity
h_w	=	vertical distance between the crest of the weir and the upstream water level
p	=	height of the crest of the weir above the bottom
Q	=	discharge passing the weir
u_w	=	the average flow velocity in the cross-section in which hw has been measured (m/s)

Test runs

Ten tests were conducted in two series. In the first series, four test runs were carried out aiming at determining the causes of damages during the monitoring period especially during the monsoon of 1995. Test conditions of first series are shown in Table-1.

Table-1: Test conditions of first series

Test	Bed Geometry	Structure			Floating debris	Flow Condition			
	Survey	G_2	G_3	G_A		Reference	Flow Attack	Water Level	Velocity (m/s)
T_1	14/06/95	Initial Design			No	18/06/95	parallel flow	21.3	3
T_2	14/06/95	Initial Design			Yes	18/06/95	parallel flow	21.3	3
T_3	14//06/95	Initial Design			No	10/07/95	oblique flow (30^0 to bank line)	22.4	3
T_4	16/07/95	Situation end of July			No	18/08/95	oblique flow (30^0 to bank line)	21.1	3

The objectives of the first series of test were as follows:

T_1: To investigate the causes of the first slide at groyne G_2 appeared and also why the bank was erroded.

T_2: To investigate the rule of floating debris trapped upstream from the piles, and especially their influence on scour hole development.

T_3: To study the influence of the flow attacking direction changing from parallel to oblique (30°).

T_4: To check whether the damages in groyne G_2 were responsible or not of the main damages of groyne G_3.

The second series of tests is a parametric study. Test conditions of second series are shown in Table-2.

Table-2: Test conditions of second series

Test	Bed Geometry	Groyne G_2	Dist. G_2-G_3 (m)	Groyne G_3	Dist. G_3-G_A (m)	Groyne G_A	Floating Debris	Flow attack	Water level	Av. vel. (m/s)
T_5	8.0 +mPWD	15^0 towards u/s, 30 m longer than as built (permeability 90%), groyne head unchanged, falling apron extended.	300	15^0 towards u/s, 30 m longer than as built (permeability 90%), groyne imp. head rounded, falling apron extended.	300	Same as T_4	Yes	30^0	22.4	3
T_6	Same as T_5	Normal to the bank, G_2 same as for T_5, groyne head unchanged, falling apron reduced.	300	Normal to the bank, G_3 same as for T_5, groyne impermeable head rounded, falling apron reduced.	300	Same as T_4	Yes	30^0	22.4	3
T_7	Same as T_5	15^0 towards u/s, total length 198 m, impermeable length 95 m, permeable length 103 m (40%, 50%, 60%, 70%, 80%), groyne head modified, falling apron slope 5:1	300	15^0 towards u/s, total length 192 m, impermeable length 75 m, permeable length 117 m (40%, 50%, 60%, 70%, 80%), groyne imp. head rounded, falling apron slope 4:1.	300	Straight bank bet. G_3 and G_A. G_A same as T4.	Yes	30^0	22.4	3
T_8	Same as T_5	15^0 towards u/s, total length 105m, impermeable length 0 m, permeable length 105 m (40%, 60%, 80%, 50%, 60%, 70%, 80%), submerged groyne, falling apron on the bank slope.	300	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (40%, 50%, 60%, 70%, 80%), plunging groyne, falling apron on the bank slope.	300	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (40%, 50%, 60%, 70%, 80%), falling apron on the bank slope.	Yes	45^0	22.4	3

T_9	Same as T_5	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (40%, 60%, 80%, 50%, 60%, 70%, 80%), submerged groyne, falling apron on the bank slope.	250	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (40%, 50%, 60%, 70%, 80%), plunging groyne, falling apron on the bank slope.	250	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (40%; 50%, 60%, 70%, 80%), falling apron on the bank slope.	Yes	45^0	22.4	3
T_{10}	Same as T_5	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (80%, 60%, 80%, 50%, 60%, 70%, 80%), submerged groyne, falling apron on the bank slope.	200	15^0 towards u/s, total length 105 m, impermeable length 0 m, permeable length 105 m (80%, 50%, 60%, 70%, 80%), plunging groyne, falling apron on the bank slope.	200	15^0 towards u/s, total length 105 m, impermeabl e length 0 m, permeable length 105 m (80%, 50%, 60%, 70%, 80%), falling apron on the bank slope.	Yes	45^0	22.4	3

The objectives of second series were to gain more information/knowledge of the behaviour of the groynes/groyne field in order to be a position to formulate the design rules and to work out guidelines and manuals for their application.

Test results and discussions

The test observations in the vicinity of G_2, G_3 & G_A for different tests in first and second series are shown in Table-3.

Table-3: The test observations in the vicinity of G_2, G_3 & G_A for different tests in first and second series

Test no.	Test observations
T_1	There was eddy downstream of G_2 with high bank current between G_2 and G_3 and small eddy was observed at the upstream of the impermeable part of this groyne. The point of maximum velocity at downstream of G_2 was 137.5 m from the bank. There was no significant scour around the groyne G_2 except near the head of the groyne. The more sever attack was observed at the connection of permeable and impermeable part of the groyne. A scour hole was developed at the toe of the falling apron. There were eddies downstream of G_3 with back currents along the bank between G_3 and G_A. At the downstream of G_3, the maximum point velocity was at 192 m from the bank line. The maximum scour was located at the extreme point of the groyne where the falling apron joined the bed. There was no displacement of falling apron during the test. At the groyne G_A which was not in the main flow, there was no significant change around the piles.
T_2	At G_2, floating debris were trapped on the whole length of the permeable groyne. There were eddies downstream of G_2 with high bank current along the bank between G_2 and G_3. There was high turbulence near the bottom around the piles. Small eddy upstream of the impermeable part was seen. The maximum point velocity was observed at 137.5 m from the bank. Scour was more or less similar as T_1. Some CC blocks were displaced to the downstream due to high vortex around the piles. The downstream part of the head of the impermeable portion of the groyne was washed out gradually. At G_3 few floating debris were accumulated in the piles. There were eddies downstream of G_3 with high back current along the bank between G_3 and G_A. The maximum point velocity downstream of G_3 was 165 m from the bank line. The falling apron did not move during the test. Around G_A, situation was same as T_1.
T_3	Eddies downstream of G_2 were increased with higher back currents than T_2 along the bank between G_2 and G_3. Small eddy was observed at the upstream of G_2. The location of maximum point velocity at downstream of G_2 was 110 m from the bank line and at downstream of G_3 it was 110m from the bank line. Scour depth was observed near the bank and velocities were greater than the previous test.

Test no.	Test observations
T_4	At G_2, the flow passed through the gap between the impermeable groyne head and the remaining piles (some piles were damaged with the impermeable part of G_2). Virtually groyne G_2 was not working. Flow directly attacked the groyne G_3. The bank downstream of G_2 was eroded and remaining impermeable part of the groyne suffered strongly during the test. A stronger eddy was observed at the downstream of G_3 with higher bank velocity. Scour pattern and other observations were same as test T_3.
T_5	Eddy upstream of G_2 and back current along the bank at downstream of G_2 were observed. Maximum point velocity at downstream of G_2 was 138m from the bank. Minimum bed level was +3.5 mPWD and the maximum net scour depth was 4.5 m. Scour was observed around the piles because of high turbulence due to floating debris. Falling apron was sufficient to protect the slope. At G_3, eddies downstream of G_3 with back current along the bank between G_3 and G_A were observed. Maximum point velocity at downstream of G_3 was at 165 m from the bank. Falling apron was stable. Minimum bed level was +4 m PWD and maximum net scour depth was 4 m. G_A was same as T_4.
T_6	Eddy upstream of G_2 and back current with high velocity along the bank at downstream of G_2 were observed. Maximum point velocity downstream of G_2 was at 165m from the bank. Minimum bed level was +0.5 m PWD. The maximum net scour depth was 7.5 m. Scour was observed around the structure because of high turbulence due to floating debris. Falling apron was sufficient to protect the slope but a slide on the slope of the falling apron from the downstream corner of the impermeable head of G_2 was also observed. Eddies were observed downstream of G_3 with back current along the bank between G_3 and G_A. Maximum point velocity at downstream of G_3 was about 165 m from the bank. Falling apron was stable. Minimum bed level was +4 m PWD and maximum net scour depth was 4 m. G_A was same as T_4.
T_7	Eddy with high velocity upstream of impermeable part of G_2 and back current with high velocity along the bank downstream of G_2 were observed. Maximum point velocity of 3.7 m/s at downstream of G_2 was at 137m from the bank. Minimum bed level was -0.7 m PWD. Maximum net scour depth was 8.7 m. There was high turbulence due to floating debris. Falling apron was sufficient to protect the slope except near the piles and

Test no.	Test observations
T_7	the impermeable head of the groyne. There were eddies downstream of G_3 with back current along the bank between G_3 and G_A. Maximum point velocity of 3.7 m/s at downstream of G_3 was at a distance of about 165 m from the bank. Falling apron was stable. Minimum bed level was +3.4 m PWD and maximum net scour depth was 4.6 m. At G_A small eddy downstream of impermeable head was observed. Flow lines slightly influenced by this groyne. Maximum flow velocity of 4.1 m/s at 165 m from the bank. Minimum bed level was +6.2 m PWD and maximum net scour depth was 2 m.
T_8	At G_2 few floating debris were trapped upstream of the groyne along the embankment. At the downstream of G_2 no back current was formed. Small amount of flow diverted by G_2, as a result G_3 was attacked with high velocity. Maximum flow velocity near the groyne was observed 2.8 m/s .Maximum velocity of 3.3 m/s along the section was at 165 m from the bank. Falling apron did not move even around the piles. At G_3 more floating debris were trapped at upstream. Back currents downstream of G_3 were minimized and the flow was parallel to the bank. Maximum velocity of 3.5 m/s was at a distance of 137.5 m. At the end of the falling apron , the slope was eroded and the maximum net scour depth was 6.5 m. At G_A large amount of floating debris were trapped which affected the groyne. These floating debris created a back current upstream of G_A but no back currents were observed downstream of G_A. Maximum velocity of 4 m/s at 137.5 m from the bank. Maximum net scour depth was of 3.9 m was observed near the piles which were not protected by the falling apron.
T_9	At G_2 few floating debris were trapped upstream of the groyne along the embankment. No back current was formed downstream of G_2. Small amount of flow diverted by G_2, so the main flow attacked G_3 with high velocity but less than test T8. Maximum flow velocity near the groyne was 2.25 m/s. Maximum sectional velocity of 2.8 m/s was observed at 220 m from the bank. Falling apron did not move even around the piles. At G_3 more floating debris were trapped at upstream. Back currents downstream of G_3 were minimized and flow was parallel to the bank. Maximum sectional velocity of 3.7 m/s was at a distance of 137 m from the bank. At the end of the falling apron , the slope was eroded and the minimum scour level was 5.5 mPWD. At G_A a number of floating debris

Test no.	Test observations
T_9	were trapped which affected the groyne. These floating debris created a back current upstream of G_A but no back currents were observed downstream of G_A. Maximum sectional velocity near the groyne was 3.5 m/s at 110 m from the bank. Minimum scour level was of 6.4 m PWD was observed near the piles which were not protected by the falling apron.
T_{10}	At G_2 small amount of floating debris were trapped upstream of the groyne along the embankment. No back current was formed downstream of G_2. Small amount of flow was diverted by G_2, so the main flow attacked G_3 with high velocity but less than test T_9. Maximum flow velocity near the groyne was 2.44 m/s. Maximum sectional velocity of 3.0 m/s was observed at 220 m from the bank. Falling apron was stable even around the piles. At G_3 more floating debris were trapped at upstream. Back currents downstream of G_3 were minimized and the flow was parallel to the bank. Maximum sectional velocity near the groyne was 3.4 m/s was observed at a distance of 137 m from the bank. At the end of the falling apron , the slope was eroded and the minimum scour level was 5.4 m PWD. At G_A large amount of floating debris were trapped which affected the groyne. Maximum sectional velocity of 3.33 m/s was observed at 137 m from the bank. Minimum scour level was 5.2 m PWD was observed near the piles which were not protected by the falling apron.

First series of test indicates that damage of G_2 was mainly due to the oblique flow and floating debris accumulated upstream of G_2. The floating debris accumulated at the upstream of the groyne reduces the opening of the upper part causes high velocities through the bottom of the pile and thus influenced to cause the damage of groyne G_2. Due to inactivity of groyne G_2 there was a severe flow attack on groyne G_3. Second series of tests resulted the formulation of design rules and parameters which will be used for permeable groynes as bank protective structures in the river Jamuna and other rivers in Bangladesh and this will be done by the FAP authority.

Conclusions

The findings of the model indicates that construction of permeable groynes are effective to protect the river bank at kamarjani area. But the structures to be strengthened to some extent based on the updated design formula and this formula could also be used for other rivers. The design formula will be updated later on by the Consulting Consortium of FAP 21/22. Bed and bank armouring should be done sufficiently to protect the damage caused by floating debris.

Acknowledgment

The authors would like to acknowledge all the officers and staff of RRI who were involved in the model study. The authors would also like to thank Mr. Eric Deubet, Mr. Denish Carrion and Mr. Manjur Kader, local consultant for consulting consortium FAP-21/22 for their active and sincere participation in the modeling work.

References

Ahmed, M. (1953) **Experiment on design and behavior of Spur-Dike**, Proceedings of IAHR, ASCE, Joint Meeting, University of Minnesota, USA.

FAP 21/22 (1996) **Bank Protection and River Training (AFPM) Pilot Project**, Test and Implementation Phase, Monitoring and Adaptation Report at Kamarjani Test Site

FAP 21/22 (1996) **Bank Protection and River Training (AFPM) Pilot Project**, Test and Implementation Phase, Technical Report No.5, Additional Model Tests at Kamarjani Site.

FAP 21/22 (1992) **Bank Protection and River Training Pilot Project**, Physical Model Tests, Draft Final Report, Planning Study, Volume-VI.

Garg, S.K (1987) **Irrigation Engineering and Hydraulic Structures**, 7th Edition, Khanna Publishers, New Delhi.

RRI (1996) **Model Investigation of River Bank Protection of Kamarjani Area Under Fap(21/22) Project**, Final Report, No.119.

Sharp, J.J. (1981) **Hydraulic Modelling**, The Butter Sworth Group, Billing and Sons, England.

Van Rijn, C.(1984) **Bed forms and alluvial roughness**, Sediment transport, part-III, ASCE Journal of Hydraulic Engineering, Vol.110, No.12, pp. 1433,1434,1736 & 1750

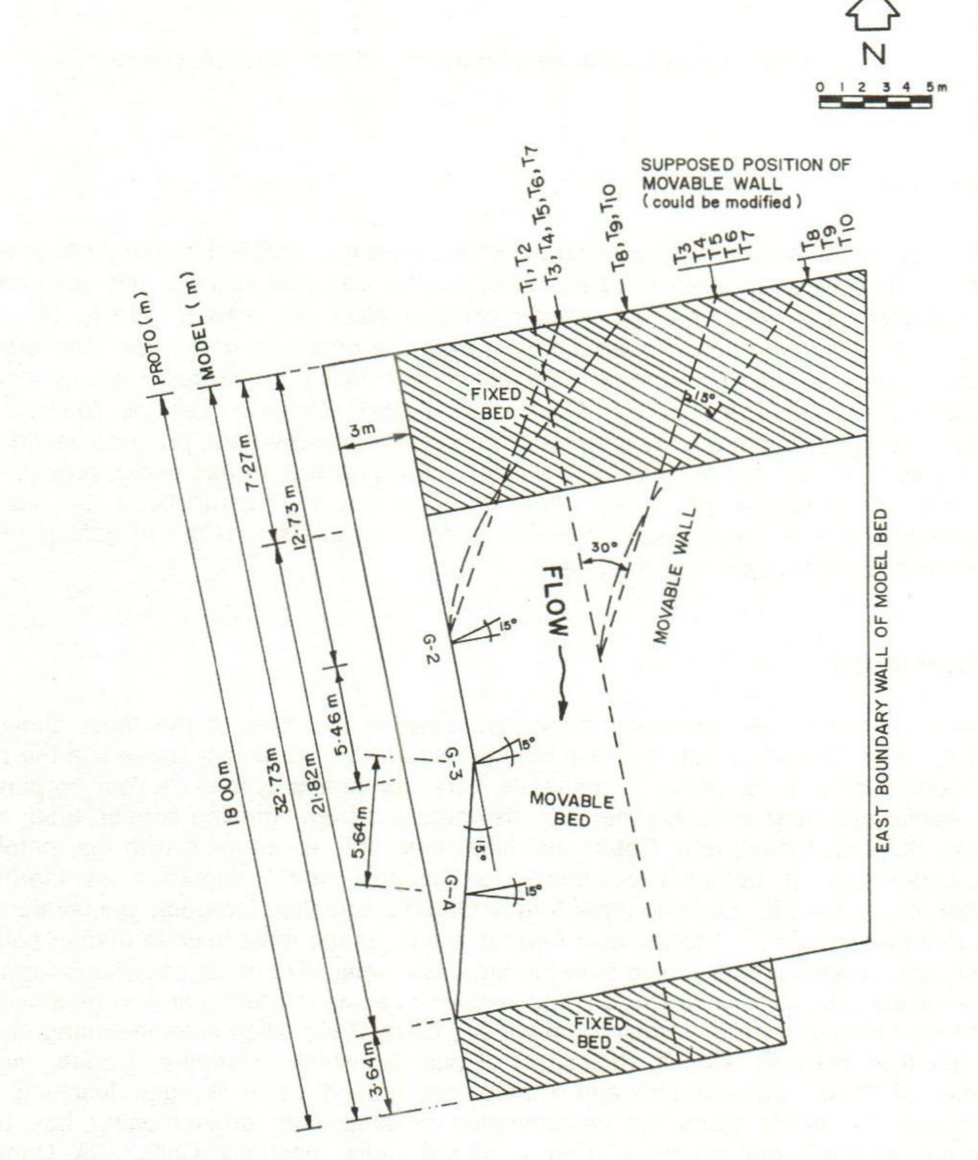


FIG:- I : MODEL DIMENSIONS

ARSENIC PROBLEM AND ITS INVESTIGATIONS

A.S. Anwarul Islam[1], Md. Nurul Haque[2] and Md. Hanif Mazumder[3]

Abstract

The arsenic contamination in drinking water is a most vital problem in many countries of the world. Millions of people of the West Bengal province in India and neighboring Bangladesh is at high risk of arsenic contamination. At present people of many countries of the world have been suffering from the problems of arsenic. The arsenic contamination in drinking water in Bangladesh is probably the largest mass poisoning case in the world. The people of Mexico, USA, Chili, Mongolia, Taiwan, Thailand are also suffering from the toxicity of arsenic. Immediate extensive and proper investigation of arsenic are very essential for the solution of this problem. In this paper, problems of arsenic, investigation of arsenic and the results of investigations by various organizations have been described. This paper also deals with the investigations of arsenic in drinking water of RRI campus.

Introduction

Arsenic is a naturally occurring poisonous element and one of the most abundant materials in the earth crust. It is tasteless and odorless. It is usually present in the form of compounds. It is found in minerals such as arsenopyrites, realgar, orpiment, arsenolite etc. near surface zone. It is also obtained from smelting copper, lead, zinc, Gold, Sulpher, Nickel, Iron, Cobalt and other ores. It is an element with the metalloid characteristics. It behaves sometimes as organic and sometimes as inorganic substances. Arsenic occurs in three forms- Organic complex, Inorganic pentavalent As^{5+}, and trivalent As^{3+}. The trivalent form of arsenic is the most toxic in human bodies. Inorganic arsenic is more toxic than the organic arsenic. The most poisonous forms of arsenic are arsenic trioxide (As_2O_3) and sodium arsenate ($NaAsO_2$). It is very difficult to remove trivalent form of arsenic from water (Dàve,1997). High arsenic contamination is found in volcanic deposit, deposit of alluvial lacustrine in semi-arid zone, mining areas of Gold, uranium etc. and comes into ground water through leaching and seepage. In recent years the concentration of arsenic in ground water has been increasing in different countries of the world e.g. India, Mongolia, Chili, USA, Canada, Japan, Argentina, Taiwan, Thailand etc. The problem of arsenic toxicity has already taken serious turn in Bangladesh. So, it is the proper time to take preventive measures and to investigate the real causes of arsenic problems.

[1]Senior Scientific Officer (In Charge), RRI, Faridpur
[2]Senior Scientific Officer, RRI, Faridpur
[3]Director (In Charge), Geotechnical Research, RRI, Faridpur

Impact of Arsenic

Arsenic is a unseen killer. Its poisoning action occurs when its intake in human body exceeds the permissible limit. The maximum permissible value of arsenic in drinking water according to World Health Organization is 0.01mg/l. In Bangladesh maximum permissible value of arsenic in drinking water is taken as 0.05 mg/l (WHO Guideline, 1997). Arsenic enters in the human body through drinking water and food. When the amount of ingestion is greater than the excretion with urine, the arsenic accumulate in hair, nails, skins and bones. Generally, arsenic poisoning in human body is found in two forms such as acute poisoning and chronic poisoning. Acute arsenic poisoning occurs in the body from the ingestion of large amount of arsenic within short time whereas chronic poisoning occurs when a small amount of arsenic enters into body through food and water over prolonged period (Khaleque, 1996).

Although small amount of arsenic is favourable for good growth of plants but it is toxic in high concentrations and excess use of arsenic insecticides causes iron deficiency (Ambasht 1990). Rice production may affect due to use of high arsenic contaminated water for irrigation.

Social impact

Effect of arsenic is also creating problem in social life. An example of West Bengal incidents may be mentioned. The arsenic affected people of West Bengal, India are facing the following problems (Duryog, 1996):

i) Wives are sent back to their parents even sometimes together with their children.

ii) In villages it becomes a headache for parents to get their affected daughters married.

iii) Even competent candidates called for interview are not offered jobs, after noticing the skin manifestation.

iv) Ignorant villagers often confused skin manifestation with leprosy and therefore avoid the person socially.

Arsenic problem

At present arsenic has been taken into account as a dangerous environment pollution element. In recent years it has been detected as a serious health risk in many countries of the world. Though arsenic has a small contribution for its usefulness in food but its presence in air and drinking water is very harmful. According to World Heath Organization (WHO) the amount of arsenic present in the environment and how much amount of arsenic enters into human body through air, food and drinking water have been mentioned in Table-1.

Table-1: Amount of arsenic present in environment and amount of arsenic enter into human body (WHO, 1996 after Akram, 1997)

Media	Arsenic level in media	Taken per day	Arsenic taken per day	Remarks
Air	0.4-30 ngm/m^3	20 m^3	0.01-0.6 μgm	May be more in Industrial area.
Food	0.4-120 μgm/kg	1 kg	0.4 - 120 μgm	75% organic As and 25% Inorganic As.
Water (normally)	1-2 μgm/l	2 litre	2 - 4 μgm	Actually, Inorganic arsenic is risky.
Water (some cases)	1200 μgm/l	2 litre	24000 μgm	Arsenic affected area.

Arsenic poisoning in human body through food is known as organic arsenic and it is not so harmful. But drinking of arsenic polluted water is very dangerous as it is inorganic arsenic. At present arsenic polluted water is at high risk of health in many countries of the world including Bangladesh. Some of the arsenic polluted countries of the world with the population affected by arsenic is shown in Table-2.

Table-2 : Arsenic polluted countries of the world (Chowdhury & others 1997 after Akram,1997)

Name of the country	Year	Population affected by arsenic	Skin Cancer (%)
India, West Bengal.	1978 - 1995	10,00,000	20
Taiwan	1961 - 1985	1,03,000	19
Chili	1958 - 1970	1,30,000	16
Argentina	1938 - 1981	10,000	much
Mexico	1963 - 1983	2,00,000	21
Thailand	1987 - 1988	14,000	18
Bangladesh	1993 - to date	23 - 25 million	Not yet estimated

Arsenic problem in Bangladesh

In Bangladesh arsenic contamination in drinking water was first detected by DPHE in 1993 and the issue came in lime light at the beginning of 1995. Now in Bangladesh about 23 million people of 42 districts out of 64 have been suffering from various complications caused by arsenic pollution (Dave, 1997; Mazumder, 1998). Some media estimates that this figure may be much higher. The high risk arsenic polluted districts of Bangladesh are Nawabgonj, Rajshahi, Kushtia, Meherpur, Chuadanga, Satkhira, Bagerhat, Faridpur, Pabna and Jessore etc. Besides high risk areas some scattered incidents of arsenic contamination has been found in Bangladesh. Such as " Narayangonj incident". The arsenic affected districts in the western border belt of

Bangladesh around the Ganges deltaic plain have been shown in Figure-1. The principal manifestation of chronic arsenic poisoning in Bangladesh include pigmentation disorders and hyperkeratosis. The awareness of the problems of high arsenic concentration in ground water has been growing since 1993. The awareness of the scale of the arsenic problem were accelerated from the International conference on " Arsenic in Ground Water " at Jadabpur University in Calcutta 1995. Since 1995 attempts are being made by various organizations to investigate the arsenic problem and to find out the solution of it.

Investigation of Arsenic

The Govt. of Bangladesh (GOB) is committed to investigate the arsenic problem and to find out the effective solution of it. For this purpose a national steering committee chaired by the Ministry of Health has been set up to direct investigations and to facilitate a mitigation programme. Sub committees comprise a Scientific Research Committee and a Technical Committee have also been formed. Representatives from Directorate General of Health Services (DGHS), Department Of Environment (DOE), the National Institute of Preventive and Social Medicine (NIPSOM), Bangladesh Atomic Energy Commission (BAEC), Department of Public Health engineering (DPHE), Geological Survey of Bangladesh (GSB), Bangladesh University of Engineering & Technology (BUET), Dhaka University (DU), WHO and UNICEF are the member of the committee (Smedley, 1997).

The priority areas for action have been highlighted by the committee as :

i) Investigation of the extent of the affected areas and sources of the arsenic.

ii) Development of appropriate technologies for water treatment.

iii) Treatment of affected populations.

iv) Development of public awareness.

Responsibility for monitoring of the quality of drinking water in Bangladesh lies with DPHE. A large number of ground water samples have been collected by DPHE and tests have been carried out on the sample for determination of arsenic concentration in their laboratories. 130 nos. of samples have been tested by BAEC and around 1200 samples have been analyzed by NIPSOM. Some water samples and 200 biological samples have been analyzed by the School of environment studies (SOES) at Jadabpur University, Calcutta. 700 samples have been collected according to the direction of Department of Geology, Rajshahi University and analyzed at SOES. The districts affected by high arsenic concentration have been shown in Fig-1. The regional extent of high arsenic water in each district has not yet been investigated in detail. So far investigation, maximum concentration of arsenic have been detected upto 2.9 mg/l and badly affected tube wells in Chapai Nawabgonj district have been sealed (Smedley, 1997). The source of arsenic is yet unknown. However, it is established as a mineralogical source with mobilization resulting from natural geo-chemical processes. High arsenic concentrations are associated with reducing ground waters rich in ferrous iron, abstracted from quaternary confined and semi-confined alluvial/deltaic aquifers.

In the badly arsenic affected areas, delineation of the scale of the problem is being tackled by analysis of drinking water by DPHE and clinical diagnosis by DGHS. DPHS is implementing a ground water sampling programme and the samples are being analysed in three of the four regional DPHS laboratories in Khulna, Rajshahi and Comilla. Health aspect of arsenic problem are also being investigated by NIPSOM and Dhaka Community Hospital. A case study for detection of arsenic in ground water at the village samta under sharsha Thana in Jessore district performed by DCH in 1997. In this connection the investigators of DCH tested water samples collected from 265 tube wells i.e., one sample was collected from each tube well. The samples were analyzed by DCH and results have been shown in Table-3.

Table-3: Arsenic concentration in drinking water of Samta village under Sharsha thana in Jessore district. (Newsletter, Dhaka Community Hospital, 1997).

Range of Arsenic Concentration (mg/l)	Number of Samples
Below 0.01	5
0.01 - 0.049	18
0.05 - 0.099	100
0.10 - 0.299	96
0.30 - 0.499	14
0.50 - 0.699	21
0.70 - 1.000	11
Total	265

DPHE is implementing the following programme by the financial support of UNICEF and WHO (Smedley, 1997).

i) Hydro-geological investigation

ii) Provision of analytical equipment and training of DPHE officials.

iii) Increasing of public awareness.

The Hydro-geological investigation incorporates a drilling programme of three cored bore holes upto 150m depth in Chapai Nawabgonj district, which has been identified as one of worst arsenic affected areas. The drilling is carried out on behalf of DPHE by a consortium headed by DU Geology Department using drilling equipment from BWDB. Geo-chemical analysis is carried out by GSB. The capability building programme of DPHE includes the provision for arsenic analytical equipment for three DPHE laboratories as well as field test for arsenic analysis of water.

Arsenic analytical capabilities in Bangladesh is limited, though some new facilities are becoming available in response to the urgent demand. Current facilities for chemical analysis of arsenic in Bangladesh include (Smedley, 1997) :

BAEC	Atomic Absorption Spectrometry (AAS) with hydride generation
DPHE	Photometric methods in three of the regional laboratories and field kits for rapid semi-quantitative analysis
DGHS	AAS
NIPSOM	Field kits and Colorimetric method
GSB	Gutzeit method

In addition to these at present RRI, JU,DU also test arsenic.

RRI	Field kit
JU	Silver Diethyldithiocarbamate method
DU	Electro-chemical method

Investigations of Arsenic by RRI

In the River Research Institute (RRI) the chemical, water pollution and ground water utilization laboratory is rendering services for analysis of surface and ground water samples for using water in different purposes. RRI is situated in Faridpur, and this area is a flood plain of the Padma river. The geological formations of this area composed of alluvial soil. The ground water of the area contains a substantial amount of arsenic (0.10-0.26 mg/l) and iron (6.10 - 9.20 mg/l) [ref. Table 4] and water of this quality is not suitable for use in domestic works as well as drinking purposes due to the presence of high iron and arsenic content (According to WHO permissible limit of arsenic and iron are 0.05 mg/l and 1 mg/l respectively for Bangladesh) (WHO Guideline,1997).

Two test borings were done/drilled for setting up a deep tube-well to supply drinking water in RRI residential quarters and offices in the campus. The samples were collected from two borings at their depths of 290 ft. The samples were tested in chemical water pollution and ground water utilization laboratory of RRI by arsenic field kit. The similar samples were also tested in Dhaka University (DU) and Jahangirnagar University (JU). The results obtained have been presented in Table-4. The most remarkable achievement of sediment, chemical, water pollution and ground water utilization division of RRI is to supply Iron and arsenic free (within permissible limit according to BSTI's guide line) water for RRI campus through a low cost Iron removal water treatment plant.

To develop a low cost technology for Iron removal from water supplied by RRI deep tube well, the chemical water pollution and ground water utilization division took up a research project. The research was carried out under the advisory guidance of Department of Environmental Engineering., BUET, Dhaka. The research was conducted in three phases by using horizontal roughing filter and up flow filter in pilot plant built for iron removal. It is mentionable here that water is supplied through the iron removal plant from running DTW-1. It has been made possible with the help of this iron removal plant to lower iron and arsenic concentration from 6.60 mg/l to 2.4 mg/l and 0.20 mg/l to 0.062 mg/l respectively as shown in Table-4.

Table 4: Comparative tests results of Arsenic and Iron of RRI Deep Tube wells (1997- 98).

Sl No	Tube Well	Name of Parameters	Tested by RRI		Tested by Dhaka University		Tested by Jahangir Nagar University		Guide line Value	
			Before Treatment mg/l	After Treatment mg/l	Before Treatment mg/l	After Treatment mg/l	Before Treatment mg/l	After Treatment mg/l	WHO mg/l	Bangladesh Standard
1	2	3	4	5	6	7	8	9	10	11
1	Running DTW-1	Arsenic	0.15 - 0.20	0.05	-	0.062	-	-	0.01	0.05
2	Proposed DTW Test Boring-2	Arsenic	0.10 - 0.15	-	0.115	-	0.26	-	0.01	0.05
3	DTW Test Boring-1	Arsenic	0.20 - 0.30	-	-	-	-	-	0.01	0.05
4	Running DTW-1	Iron	6.60	2.40	-	-	-	-	0.3	1.0 (urban area) 5.0 (Rural area)
5	Proposed DTW Test Boring-2	Iron	9.20	-	-	-	-	-	0.3	- Do -
6	DTW Test Boring-1	Iron	6.10	-	-	-	-	-	0.3	- Do-

Discussion

Arsenic level in different media and how much amount of arsenic enter into human body from different media have been shown in Table-1. From this table one can get an idea about the arsenic level and amount of arsenic taken per day from different media. In Table-2 the population affected by arsenic of different countries of the world including Bangladesh has been shown. This will give information about how much people of the world are affected by arsenic contamination. The result of arsenic content obtained by DCH by the analysis of water samples collected from different tube wells of Samta village under Sharsha thana of Jessore district have been presented in Table-3. Out of 265 tube wells, water of only 5 tube wells (2% tube wells) was found to be safe for drinking as per WHO recommended arsenic content in drinking water (0.01 mg/l) and water of 18 tube wells (7% tube wells) only contained arsenic upto the maximum permissible limit 0.05 mg/l. Water of 242 tube wells i.e., 91% of the existing tube wells were unsuitable for human uses. (DCH, 1997). Comparative tests results of arsenic and iron concentration in water samples collected from running deep tube well (DTW-1), DTW test boring no.-1 and DTW test boring no.-2 in the RRI campus, tested by DU, JU and RRI have been shown in Table-4. From this Table one can get an idea about iron and arsenic content in the drinking water of RRI campus. The result of iron concentration in water samples of running DTW-1 tested by RRI before and after treatment through the iron removal plant has also been shown in same Table. It can be seen from the Table that the results of iron concentration before treatment is beyond the permissible limit of BSTI guideline value and that of after treatment is within the permissible limit of BSTI guideline value. Investigations of arsenic by different organizations have been discussed in this paper. These will give the idea about organizations which are engaged in the investigation of arsenic, problems associated with arsenic and solution of arsenic problem in National level.

Conclusion and recommendation

Arsenic Contamination in drinking water is a serious problem in Bangladesh. Immediate steps should be taken for investigations of arsenic in drinking water all over Bangladesh. The people residing in arsenic polluted area or danger zone should not drink tube well water directly; rather the water may be used after preserving for a full night and also filtering. Both the numbers of people affected by arsenic related problem and the areal distributions of high-arsenic in drinking water should be identified clearly. The subsurface geology and hydro-geology of the alluvial and deltaic aquifers of affected area should be further investigated in order to identify mineral sources of arsenic and mobilizations pathways. Appropriate 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 can play a vital role through advertising to aware the people about the adverse effect of arsenic. The research organizations which have the facilities for analysis of arsenic should take up " Research Project / Programme" to solve the arsenic related problem. A national data base center should be installed for collection of information, preservation, exchange and distribution of data regarding arsenic problems.

References

Akram, S. (1997) **Arsenic problem**, published in the Daily Sangbad on 31st August, 1997.

Ambasht, R.S. (1990) **Environment and Pollution (An Ecological Approach)**.

Dave Prof. J.M (1997) **Assignment report on arsenic contamination of Drinking water in Bangladesh**.

Dhaka Community Hospital (1997) **Monthly newsletter (June)**, Volume-8, Issue-2.

Duryog Nivaron (1996) **Newsletter from Bangladesh**. A publication of Disaster Preparedness forum Volume -V.

Gorcheve, H.G. (1997) World Health Organization, **Guideline value for arsenic in drinking water**.

Khaleque, S.A. (1996) **Arsenic Poisoning**, published in The Independent on the 28th October, 1996.

Mazumder, M. K.I (1998) **Arsenic Menace**, A case of mass poisoning, Published in The Independent, on 20th February, 1998.

Smedley, P.L. (1997) **Arsenic in Ground Water in Bangladesh**, Report of Study Visit, British Geological Survey, 11 - 28 February, 1997.

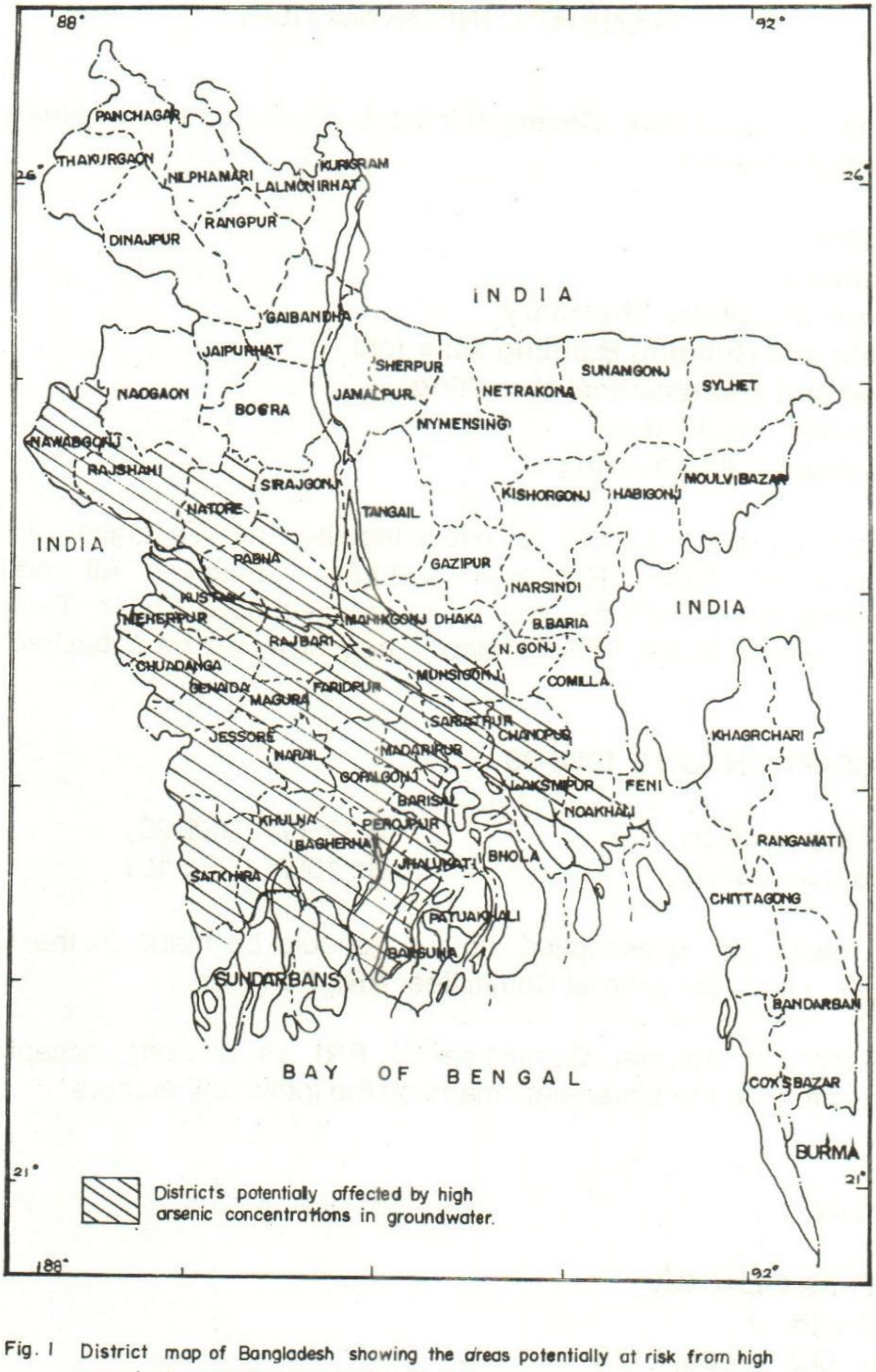


Fig. 1 District map of Bangladesh showing the areas potentially at risk from high groundwater-arsenic concentrations. (Ref : Report of a study visit by P L Smedly, 1997)

GENERAL INFORMATION

Technical journal of River Research Institute covers scientific research articles in the fields of

Hydraulics
Geotechnics
Sediment and Water Chemistry
Concrete and Building Building Materials
Physical and Mathematical Modelling
Ground Water Utilization
Environmental Engineering

Technical Journal of RRI is published by the Technical Journal Committee of River Research Institute, Faridpur. All editorial correspondence should be directed to the Executive Editor, Technical Journal Editorial Board, RRI, Faridpur. The journal will be published half-yearly.

SUBSCRIPTION RATE 1998-99

Annual subscription :	Tk. 250/= (US$ 20)
Individual subscription :	Tk. 150/= (US$ 10)

All payments and subscription inquiries should be made to the Office Manager, Technical Journal Committee, RRI, Faridpur.

Printed by :

Khan Art Block
Niltuly, Faridpur
Phone : 0631-2685, 2173
E-mail : kmsalim@bdonline.com

NOTE : DISCUSSION FOR ALL ARTICLES WILL REMAIN OPEN UPTO APRIL 30, 1999.

[illegible] Butler Swaith [illegible] England.

[illegible] Rijn, [illegible] (1984) Bed forms and alluvial roughness, Sediment transport, part III, ASCE journal of Hydraulic Engineering, Vol.110, No.12, pp. 1433-[illegible]