RIVER TRAINING STRUCTURES IN THE BRAHMAPUTRA RIVER IN BANGLADESH, COMPARISON OF SCOUR, STUDY AND NATURE

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ABSTRACT

This paper presents results of scour around different types of river training studies supported by physical model tests and assessment of and comparison with surveys of the scour in nature for the same structures. The studies were performed by the author as team member of the River Training Studies of the Brahmaputra River (BRTS), Flood Action Plan Project No 1 (FAP 1) in Bangladesh 1990-93. The author at the time was working for DHI that was in association with Sir William Halcrow & Partners Ltd and two local Bangladesh companies EPC and DIG. The River Research Institute (RRI) in Faridpur was nominated to carry out the physical modelling studies under the guidance of the Consultant.

1. INTRODUCTION

The author has mainly over the last 48 years worked with marine and coastal engineering studies and breakwaters but worked 30 years ago for more than 2 years as hydraulics expert on the BRTS study in Bangladesh. He took interest in the hydraulics, morphology, and river training works (RTW) of this very large river. It is, with a 100-year discharge of 100,000 m^3/s, one of the largest truly braided rivers in the world. The author took over in 1991as hydraulic expert on the BRTS study from Mutlu Sumer. The study resulted in the design and construction of very important RTW works. Also, the author worked about 10 years ago as independent design checker for the RTW works on the Padma Multipurpose Bridge SW of Dhaka, where a lot of the BRTS experience could be brought to bear. The studies were made in close collaboration with my colleague, Nadia Genovese, that since then has taken deep interest in scour, both for river structures but especially for offshore wind foundations, /10/. The Padma RTW was designed by NW Hydraulics and the author had long and fruitful involvement with the designers, especially with Knut Oberhagemann. The time scale of morphological changes in the large rivers of Bangladesh is several years, and it is now 30 years since the BRTS project, and therefore the author found it interesting to revisit the studies. In this connection it was relevant to revisit the assumptions and study results and compare them to available data from the sites where experience from nature have been gained (/5/ and /6/), showing how the RTWs constructed have performed. The paper thus presents selected significant results of the BRTS studies and comparison with prototype performance.

The Brahmaputra Right Embankment (BRE) in Bangladesh is an earth embankment constructed in the sixties and seventies along the west bank of the river. The safety of the BRE and consequently the area protected by the BRE were seriously threatened by continued bank erosion from an overall trend for the river to move westwards for more than a decade. This resulted in extensive damage when breaches occurred with large scale human suffering. In March 1990, after the disastrous ~100 year flood of 1988, the Bangladesh Water Development Board (BWDB) appointed the Consultants to undertake a three-year study. The study showed it to be a priority to assure that the Brahmaputra did not break into the Bengali River; and therefore, the study included the Kalitola Groyne strengthening and the new Sariakandi and Mathurapari large hard points a few km south of Kalitola. Further, large focus was on securing the Sirajganj town, and as a result the new upstream groyne-head was studied and designed.

2. FAP 1, BRAHMAPUTRA RIVER TRAINING PROJECT

In order to prepare a master plan for the stabilisation of the riverbank and thereby improve the performance of the right bank flood embankment, it was first necessary to develop an understanding of the behaviour of the river and determine the values of key parameters that dictate the form and properties of appropriate stabilisation measures.

At the outset of the study, relatively limited details were known about the river. Cross-sections of the river had been surveyed on an annual basis since 1964, but no comprehensive analysis of this data set had been undertaken. The Jamuna Bridge Study, /2/, undertook a substantial amount of field investigation and derived valuable relationships, but this work was mainly focused on the bridge site and did not address the broader issues of bank stabilization along the whole Brahmaputra River in Bangladesh.

The BRT Study was therefore planned to build on this earlier work and extend it to cover the full 200 km stretch between the Teesta and the Hurasagar river confluences. River surveys undertaken during the 1990 and 1991 monsoon seasons and the intervening low flow period provided important new information on the form of sediment transport and the channel geometry changes that take place seasonally. Through morphological studies, insight was gained into the processes that shape the river and the relationships between the rate at which bank erosion and char (islands in a braided river) building takes place. Use of lD and 2D mathematical modelling provided further understanding of the processes and, equally important, the latter generated new data time series. These could be statistically processed to give a measure of the frequency distribution of the values of parameters (such as water level and velocity) that are of paramount importance to the designer of stabilisation works. The third key parameter for design is scour depth at man-made structures and physical modelling was undertaken to simulate 3D effects, and played an important role, in conjunction with the other studies, to establish the pattern of scour associated with different forms of training works end and thus the criteria for the design of revetments and toe protection (incl. the behaviour of falling apron).

3. ABOUT THE BRAHMAPUTRA RIVER

The study area extends 200 km southwards along the western bank of Brahmaputra (Jamuna) river from the outfall of Teesta River to the Hurasager River about 20 km north of where the Brahmaputra has its confluence with Ganges (Figure 1).

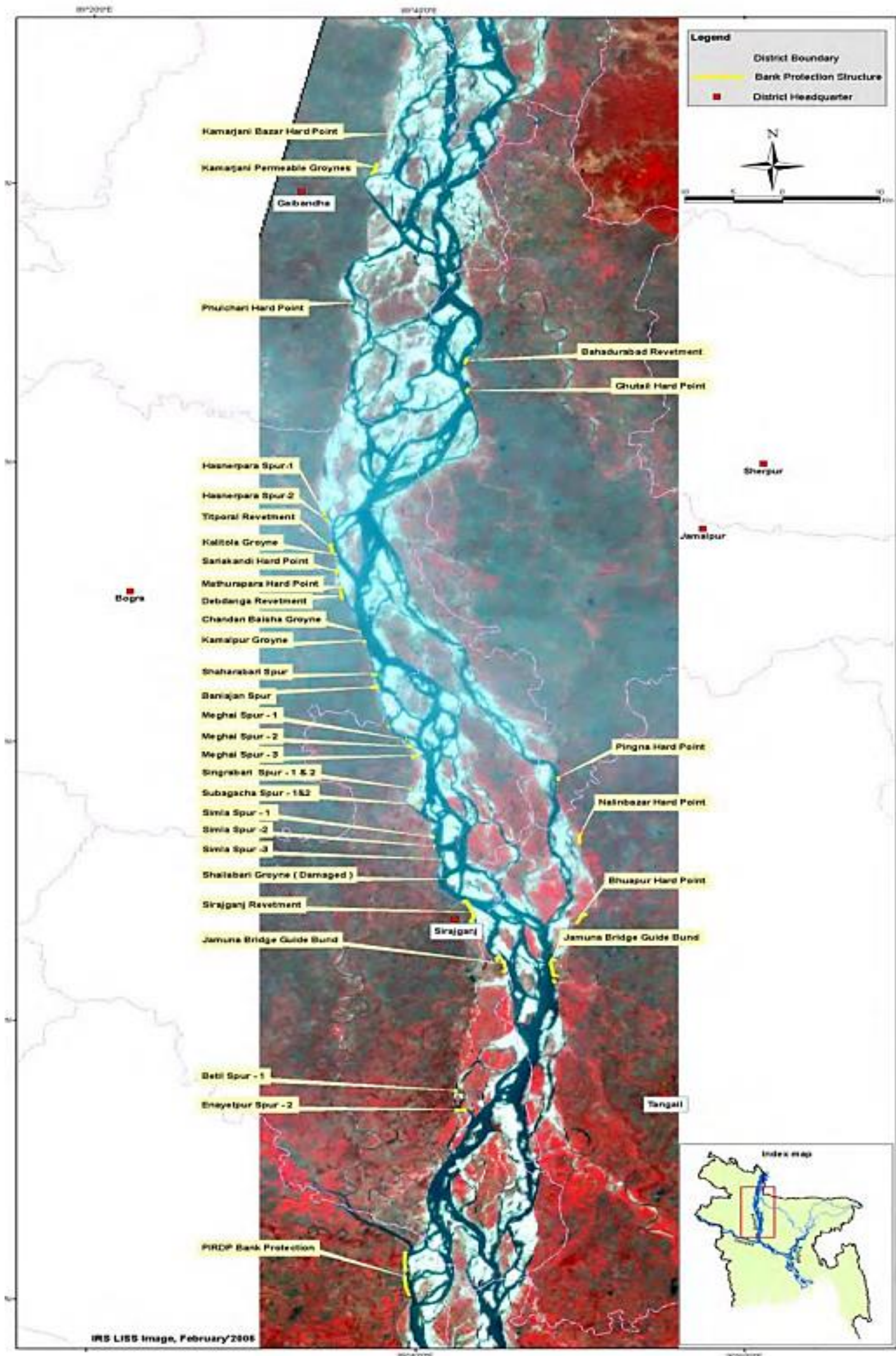


Figure 1 Location Plan, characteristic braid pattern and important locations of bank protection structures, /6/.

The objectives of the study were to develop a master plan for protection of the existing ~200 km embankment (BRE) as a long-term measure, and design of bank protection works at critical sections for priority implementation.
The Brahmaputra-Jamuna river system is one of the largest in the world, and is also the largest and most important river system in Bangladesh, accounting for more than 50 per cent of the total inflow into Bangladesh from all cross-border rivers. The river system is braided and inherently unstable. Evidence until 1990 indicated that morphologically the river has a gradual westward trend and is in general becoming wider. The pattern of bank erosion appears to arise from a combination of at least four major processes such as: evolution of major aggressive bends, systematic erosion of the bank line due to widening of the braided belt, shifting of the centre of inter-island crossovers and larger scale movement of the centreline of the braided belt. In the project area, the width of the river ranges between 7 and 15 km.
The area of the river basin is floored by Quaternary alluvial deposits, mainly sand which on the average has a grain size of 0.18 mm. The dominant discharge of the river is about 38,000 m^3/s and the 100-year discharge is about 100,000 m^3/s. The average river slope is fairly constant in the project area at $7*10^{-5}$ (i.e., a fall/slope of 7 cm/km).

4. NATURAL FLOW VELOCITIES AND DESIGN VELOCITIES FOR STRUCTURES

Traditionally, river training structures along the Brahmaputra River have been designed based on an assessment of the local site-specific conditions. The Consultants were faced with having to develop a more general understanding and knowledge of the hydraulics of this highly erratic and volatile braided river that shifts channels, develops and again destroys the char islands in the braided belt. The study had to be relatively general as the results would have to be applied to several sites along the western bank of the river. The study arrived at the conclusion that within the project area, the river cross-sections and associated flow characteristics can, as an approximation, be considered as independent events belonging to the same statistical/stochastic process. In other words, the development of the river and its channels can be treated by statistical methods. The time scale of these major changes is so short, say an estimated one to two decades, that within the lifetime of the structures, there is a probability that the worst conditions in terms of water depth and flow velocity found anywhere within the 200 km stretch of the river in question can reproduce itself at any section of manmade structure introduced along the west bank of the river. The studies consequently concentrated on deriving these conditions and the associated probability functions.
In the Jamuna Bridge Study, /2/, regime equations were derived for individual channels for both high and low flow stages. There was close agreement between the two sets of equations, from which it was concluded that the channels could adapt very rapidly to changed flow circumstances.

$$Y=0.23\,Q^{0.32} \quad \text{(Eq. 1.)}$$
$$B=16.1\,Q^{0.53} \quad \text{(Eq. 2.)}$$

where Y = mean depth, B = water surface width and Q = bank full discharge.

Analysis of a limited number of cross-sections has shown that on the average the maximum depth in a cross-section is 2.3 times the mean depth. Note that the depths cited here are relative to the dominant discharge (approximately 38,000 m^3/s). For design purposes, the 100-year water level is used. This corresponds to a discharge of about 100,000 m^3/s and the corresponding water level is approximately 3 m higher than for the dominant discharge.

For model studies of scour, the water depth in an area near the structure is required. The average depth of a single channel for a 100-year situation can by Eq. (1) be estimated at Y = 9.2 m. It is clear that neither the average depth of 9.2 m which includes flooded char lands (islands) nor the absolute maximum water depths of 25 to 30 m (depending on probability of exceedance) is representative for the typical maximum average water depth that can develop near man-made structures. Based on the collected cross sections, it is possible that the deep section of a major river anabranch has depths in the range 10 to 20 m. Therefore, D = 15 m has been used as the undisturbed normal depth without RTWs in more overall RRI physical model studies of scour. This was used for the general study of Sirajganj Town Protection, while for the study of local scour, which involves a smaller area due to the larger model scale of the physical model, a water depth of 20 m has been used for testing.

This is rough assumptions as the river is so volatile that one cannot fully understand what can happen at a given location in the many years of service life of man-made RTWs introduced.

The study finally resorted to making best possible statistics of natural water depths measured in about 100 cross sections along 200 km of the river, Table 1.

Table 1 Statistics of maximum natural depth found in about 200 Brahmaputra River cross-sections and associated probability of exceedance.

Probability of exceedance (per cent)	Maximum depth below WL for dominant discharge, (m)	Maximum depth below WL for 100 year discharge (m)
90	11.5	14.5
50	13.5	16.5
20	16.5	19.5
10	18.0	21.0
5	20.0	23.0
2	24.0	27.0
1	25.5	28.5
0.5	27.5	30.5

The BRTS study and other studies on the Brahmaputra including JMBS have shown that large dunes migrate on the riverbed. The maximum height of dunes was found to be about 4.0 m. The length of the dunes is typically 100 to 200 m, and they migrate downstream with a speed that can reach about, V =1 mm/s or 3.6 m/per hour or about 90 m per 24 hours.

Therefore, any instant sounding in the river might be associated with uncertainty in the measured depths with up to ±2.0 m. This aspect also needs to be considered in the design of river training and bank protection structures and selection of maximum design water depth.

In regard to the flow pattern, the river is highly braided within the study area with complex flow patterns and horizontal velocity distribution. The design of structures has been based on the maximum near bank velocity with 100-year return period. Design velocities were derived in the following manner, Figure 2.

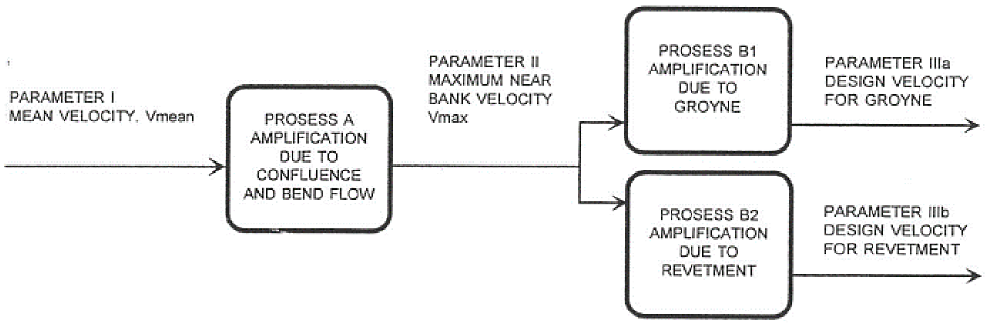


Figure 2 Parameters and processes for assessment of design velocities

The procedure steps are as follows. (a) Parameter I, Vmean, was found from an assessment of the distribution of mean velocities in independent river cross-sections for the 100-year flow condition. The distribution of Vmean was derived based on discharge data from BWDB and from MIKE 11, lD modelling of the river. The analysis comprised about 100 cross-sections with an average spacing of about 2 km. The mean value of the velocity was determined at, Vmean= 1.46 m/s with a standard deviation of 0.43 m/s. (b) Process A is the amplification of the flow velocity due to confluence or bend flow or a combination thereof. The distribution of the amplification factor was estimated from physical modelling of a number of stretches of the river that represented both “extreme” and “average” amplification. On this basis, the mean amplification was estimated at 1.50 with a standard deviation of 0.28. (c) Parameter II is the resultant maximum velocity for undisturbed flow (i.e., no man-made structures present). The mean velocity is 2.25 m/s and the ten and five per cent exceedance velocities are 3.2 and 3.6 m/s respectively. (d) Processes BI and B2 are the amplification of the flow velocity near structures for groynes and revetment structures respectively. The amplification at structures was derived from detailed measurements in the various flow models. For groyne noses, the amplification was found to be 1.40 with a standard deviation of ten per cent, i.e., 0.14. For the BRTS type revetments (hard points), the amplification was found to be 1.30 and a standard deviation of ten per cent was assumed, i.e. 0.13. (e) Parameters IIIa and IIIb. The resultant near bank velocity in the case of structures could thus be estimated as shown in Table 2.

Given a design criterion that there should be no more than a one per cent risk of the design velocity being exceeded during a 30-year nominal structure life, the values were interpreted as follows.

For a project life of 30 years, there is a probability of 26 per cent for the occurrence of a 100-year hydrological event. Thus, for a risk of one per cent over the project life, the velocities in Table 2 for eight per cent exceedance probability should be used (i.e., 0.08*0.5*0. 26 = 1%). The factor of 0.5 being applicable because the maximum velocity can only occur near one or the other bank (or an island). Table 3 shows the derived design velocities.

Table 2 Maximum velocities (m/s) for given exceedance probability for a 100year return period.

Parameter	Amplification factor	Exceedance probability (per cent)					
		2	5	8	10	20	50
U-mean (m/s)		2.3	2.2	2.1	2.0	1.8	1.5
U-max (m/s)							
No structure		4.0	3.6	3.3	3.2	2.8	2.2
Groyne	1.4	5.7	5.1	4.8	4.6	4.0	3.0
Revetment	1.1	4.5	4.0	3.7	3.6	3.1	2.3
Revetment	1.2	4.8	4.3	4.1	3.9	3.4	2.6
Revetment	1.3	5.2	4.7	4.4	4.2	3.7	2.8

Table 3 Design velocities for different structural components (based on a one per cent probability of exceedance of design conditions in the project life of 30 years).

Type of structure	Amplification factor	Design velocity (m/s)
Revetment straight section	1.1	3.7
Revetment up-stream termination	1.3	4.4
Head of groyne	1.4	4.8

5. CONCEPT OF RIVER TRAINING (RTW) STRUCTURE PARAMETERS INFLUENCING SCOUR DEPTH

The studies of a master plan for protection of the BRE resulted in a first priority to prevent the river from merging with the Bengali river flowing parallel with the Brahmaputra River. It was concluded that the most feasible way of stopping the river from moving further westward would be to introduce a system of large man-made hard points including strengthening the existing Kalitola groyne, to build new hard points at Sariakandi and Mathurapara and strengthening of the RTW at Sirajganj town. The concept for these hard-points was developed through extensive studies and finally took the form of hard-points constructed for each about 8 km distance along the river. The hard-points are situated at the average about 1.5 km in front of the BRE and the hard-points are to be connected to the BRE by crossbars. The crossbars which have a light surface protection on the side slopes are very essential structural elements to ensure that water flow behind the structure is stopped and the hard points cannot therefore be eroded from the back side. The studies of the planform of river bends have shown that with the 8 km distance between the hard-points the setback distance of 1.5 km to the BRE is adequate to allow for the natural formation of embayment between the hard-points without seriously endangering the BRE.

On this basis, the concept of the river training structures was formulated. The main task was then to design the structures in such a way that they would be able to withstand the environmental conditions likely to occur at the specific site.

6. SCOUR PRINCIPLES

The river training structures that were built in the Brahmaputra as part of the BRTS programme for Priority Works and the long-term master plan for protection of the BRE relies on an assessment of the maximum water depth likely to occur within the lifetime of the structures. The maximum depth, i.e. the lowest bed levels that are likely to occur, is dependent on the following factors: (a) normal variations in the channel cross-section geometry that occur in the course of a normal hydrological year, irrespective of any change in the channel plan geometry, (b) variations in the channel plan geometry such as the development of an aggressive bend, char approaching the bank, or anabranch confluence characteristics, (c) passage of bed forms (dunes), and (d) local scour produced by the structure itself due to its distortion of the flow field.

Data that is available for this purpose is derived primarily from BWDB surveys, the BRTS surveys and the Jamuna Multi-purpose Bridge studies (JMBS), /2/. With respect to local scour produced by the structures, data from literature and the physical and mathematical model studies performed by BRTS were included in the analysis.

6.1 Local scour

Local scour in an alluvial river is produced by the distortion and related amplification of the current pattern around an obstacle to the flow such as natural hard-points or man-made structures: groynes, revetments, bridge piers, etc. The increased velocity results in increased sediment transport capacity and the riverbed is adopting by scoured until a new equilibrium is established in which there is balance between incoming and outgoing sediment to an area.

Different researchers have developed formulae for estimating scour depths at the nose of structures. There is naturally an appreciable range in their prediction results, and they all have an uncertainty attached as no two rivers or structures are identical.

6.2 Formulae and field data

Of the formulae available, the one proposed by Liu (1961), /1/ was adopted by BRTS. It predicts scour depths in reasonable agreement with measured scour depths in small scale experiments performed by the US Army Waterways Experimental Station, Copeland (1983), /3/.

Liu’s formulae for a solid groyne with sloping sides reads:

$$\frac{y_s}{\bar{y}} = \alpha \left(\frac{L}{\bar{y}}\right)^{0.4} F^{0.33} \qquad \text{(Eq 3)}$$

Where α = constant = 1.1, ys = scour depth below mean bed level (m), y= mean flow depth, upstream (m), L = length of groyne, measured perpendicular to the bank (m), $F = V/(gy)^{0.5}$ = Froude number, and V = average velocity upstream (m/s).

7. SCOUR DEPTH STUDY FOR SIRAJGANJ AND COMPARISON WITH PROTOTYPE MEASUREMENTS AND MODEL TEST RESULTS

7.1 General on scour patterns

Prototype observations from the Brahmaputra River and model studies by BRTS show, as will be documented in the following, that for groynes and other river training structures with sloping sides, the scour hole develops almost always at a certain distance away from the structure itself. This finding although it complicates the assessment of scour depths, shows that it is not necessarily the maximum scour depth that should be applied for design of structures, but rather the maximum water depth likely to develop at the very toe of the structure in question.

7.2 Prototype observations at Ranigram groyne in Sirajganj

In 1991 only a few major structures had been built along the Brahmaputra and fewer have survived long enough to provide data useful for the assessment of local scour around structures.
One exception is the structures at Sirajganj town built in the mid 1980's. These structures are the Ranigram Groyne and the revetment type structure along the town riverbank south of the groyne.
Figure *3* shows a survey at Sirajganj from 1986 and reported in the JMBS (May 1990) report. The discharge in 1987 is assumed by JMBS to be 60,000 m^3/s, the initial water depth, lb= 13.5 m, and the flow velocity, v =2.0 m/s. This velocity is used as the undisturbed upstream velocity in the following.

It is interesting to compare the Aug '86 and March '91 situation at Ranigram Groyne, Table 4. At that time the groyne was a single groyne structure sticking out in the river in the direction West-East. Although the maximum water depth relative to the 100-year WL was 40.5 m and 31.8 m respectively, the scour pattern is somewhat similar. However, in Aug '86, the scour hole is to the south of the centreline of the groyne, while it was at the centreline in March '91. This difference can be explained by a difference in the angle of flow relative to the alignment of the groyne, with the August '86 situation being more inclined anti-clockwise due to a deeper embayment to the north at that time.

Table 4 Maximum water depth measured at Sirajganj

Date	Place	Location	Depth (m)	Depth relative to 100 year WL (15.75 m) (m)	Source
Nov '85	Sirajganj	Not known	35 below +14.0	36.8	JMBS
Aug '86	Sirajganj	at Ranigram	38 below +13.3	40.5	JMBS
Aug '86	Sirajganj	Along bank protection	30 below +13.3	32.5	JMBS
19 March '91	Sirajganj	At Ranigram	16 relative to datum PWD	31.8	BRTS

For the Aug '86 situation, the following data applies, y_s = 13.5 m, y =40.5-13.5 =27 m, L=500 m (assumed effective length of the groyne). The coefficient, α, can then be found.

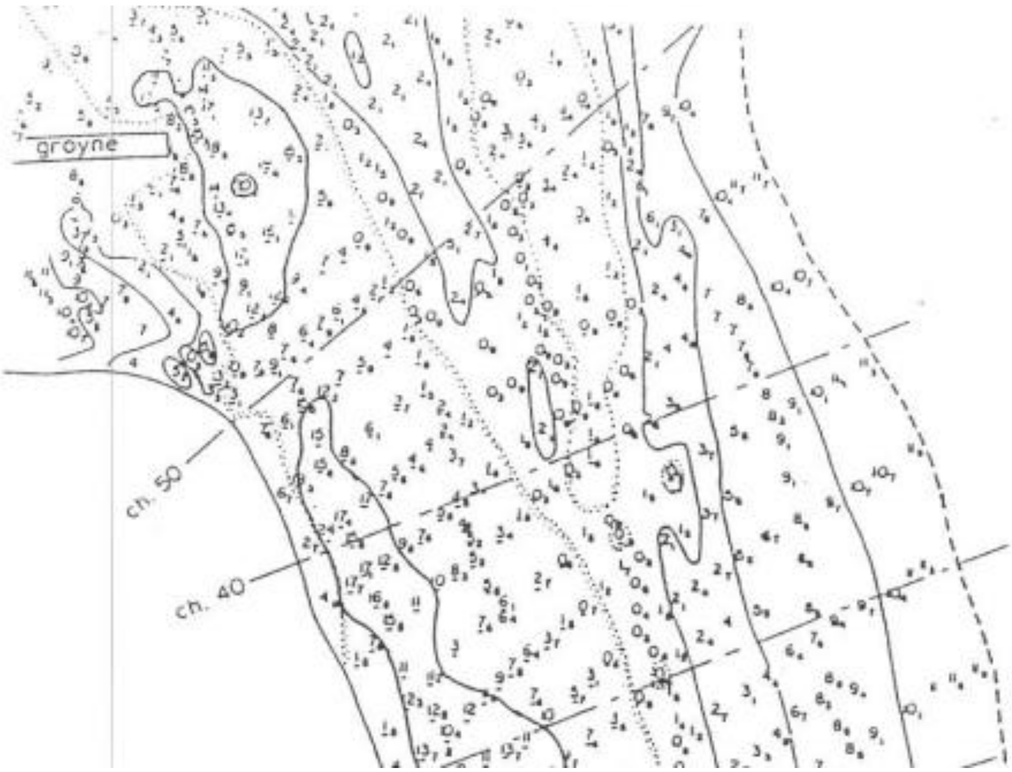


Figure 3 River survey at Sirajganj, 1986. Note the Ranigram Groyne to the north (groyne).

The value of α is 0.85 which is about 30 per cent below the value of 1.10, found by Liu´sformula. For the March ’91 situation, the scour is less, probably because these measurements were performed towards the end of the dry season. It was assumed that such scour holes can develop in a period of a month or two depending on the flow velocity and thus the sediment transport capacity. This is also documented by the fact that Ranigram groyne was built in 1985 and already in August ’86 after one year the very deep scour hole had developed at its nose.

Hotopp et al has in /5/ studied how fast the scour can develop in nature and found that the maximum recorded in Bangladesh was at Ranigram Groyne and it was -15 m in one season. Deep water was also found near the Jail Khana along the revetment and south of the groyne with maximum water depth of 32.5 m according to JMBS reports.

It appears that the maximum depth is at a distance from the toe of the structure.

It is further of interest to note that the distance from the head of the groyne to the Aug ’86 scour hole was about 200 m and that the difference in level over the outer 150 m was about 25 m, i.e., a slope of about 1:6 of the riverbed towards the scour hole.

In the BRTS soundings from March ’91, the scour hole was about 200 m away from the top of the groyne crest, i.e., about 150 m from its toe. The slope from the groyne towards the deep scour hole is about 1:5. This therefore represents a more severe condition with regards to the stability of the structure despite the lower maximum depth.

7.3 Model test of strengthening of Ranigram Groyne in the Sirajganj town protection

Three different layouts were tested, including the existing Ranigram Groyne configuration and two different layouts involving a 100 m or 200 m spur towards north respectively (Figure 4).

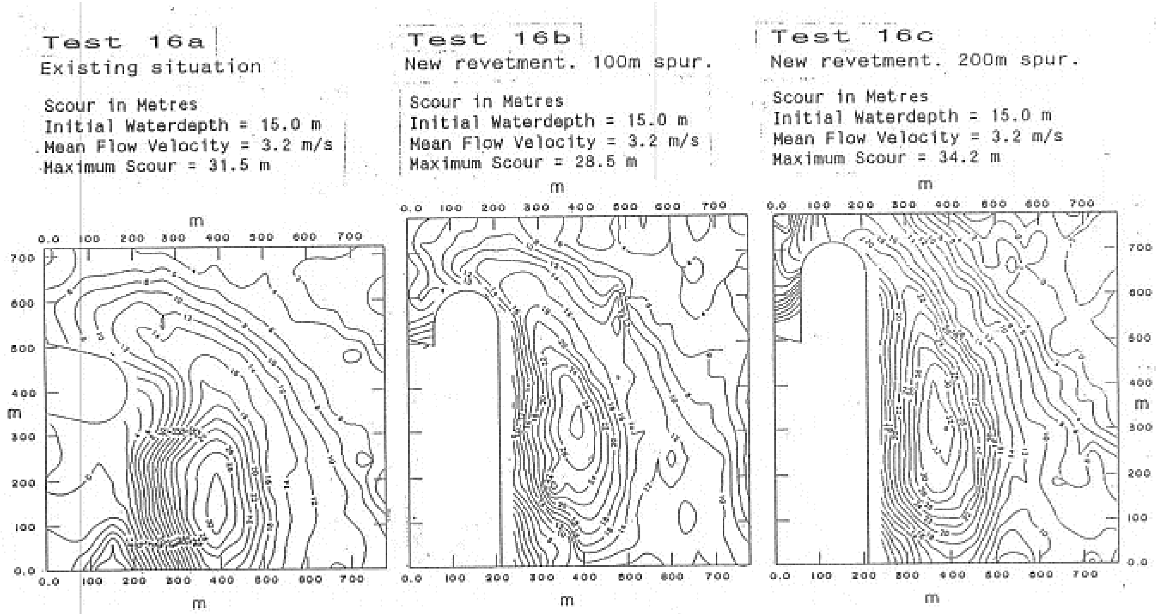


Figure 4 Scour model tests for Sirajganj town protection (the plans show scour in (m)

The tests were performed in a movable bed model with a horizontal length scale of 1:300 and a vertical scale of 1:120. The reason for selecting a distorted model was limitations of the outdoor model facility in terms of size, but especially with maximum discharge of pumps (up to approx. 500 l/s), The distorted model was used despite the knowledge, that a distorted scour model is not in similitude with nature and thus violates the model laws including the Reynold´s number effects. Anyway, As explained above the team had to resort to this type of tests for space and model water discharge reasons at RRI. The first test was the one to the left in Figure 4, with the existing Ranigram Groyne where scour observations were available from nature, see Table 4 with a total depth of 40.5 m. The model gave somewhat larger scour at 46.5 m on which basis this type of model test was judged conservative for studying also modified layouts to the groyne end, see Figure 4. The results of scour measurements after equilibrium were reached are shown in Figure 4 for three tests. It appears clearly that in all three situations, a deep scour hole developed but that the position and maximum depth of scour was not the same. The layout with 100 m spur towards north was selected for implementation.

The conditions found in nature and the model are compared with the Liu formulae. The following data apply:

For the Aug. 86 situation, the following data applies, y=13.5 m, y_s=40.5-13.5=27 m, L=500 m (assumed effective length of the groyne).

$$F = \frac{2.0}{(9.81 \cdot 13.5)^{0.5}} = 0.17 \qquad \text{Eq (4)}$$

The coefficient, α, can then be found:

$$\alpha = \frac{\frac{y_S}{\bar{y}}}{\left(\frac{L}{\bar{y}}\right)^{0.4} F^{0.33}} = \frac{\frac{27}{13.5}}{\left(\frac{500}{13.5}\right)^{0.4} 0.17^{0.33}} = 0.85 \qquad \text{Eq (5)}$$

The scour hole found in the model for the layout with 100 m spur is located downstream of the spur head and the depth is reduced by 3 m relative to the existing situation. In both cases with a spur, the distance from the straight revetment section to the centre of the scour hole is about 175 m and the slope is in the order of 1 :5 to 1 :7. It appears that although the maximum water depth including scour reaches values between 40 and 50 m, the maximum water depths near the toe (within 50 m distance) are between 27 m for the existing situation and 31 m and 35 m for the two alternative remedy solutions. The 100 m spur was selected for implementation, and the design scour depth has been taken as 34 m at the roundhead and a short stretch downstream to allow for the variation in water depth due to migrating dunes. The final layout appears in

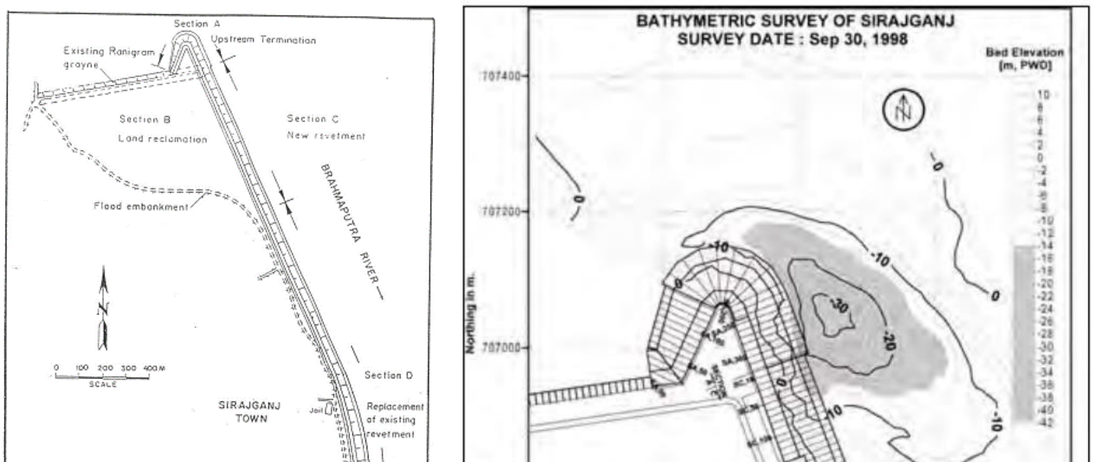


Figure **5** and the recorded scour depth of 30 m measured in Sept. 1998 and reported in /6/ is to be compared with 28.5 m found in the model tests, Figure 4. The location of the scour hole is different, which most likely can be attributed to that the upstream riverbed and embayment must have been different in model and nature.

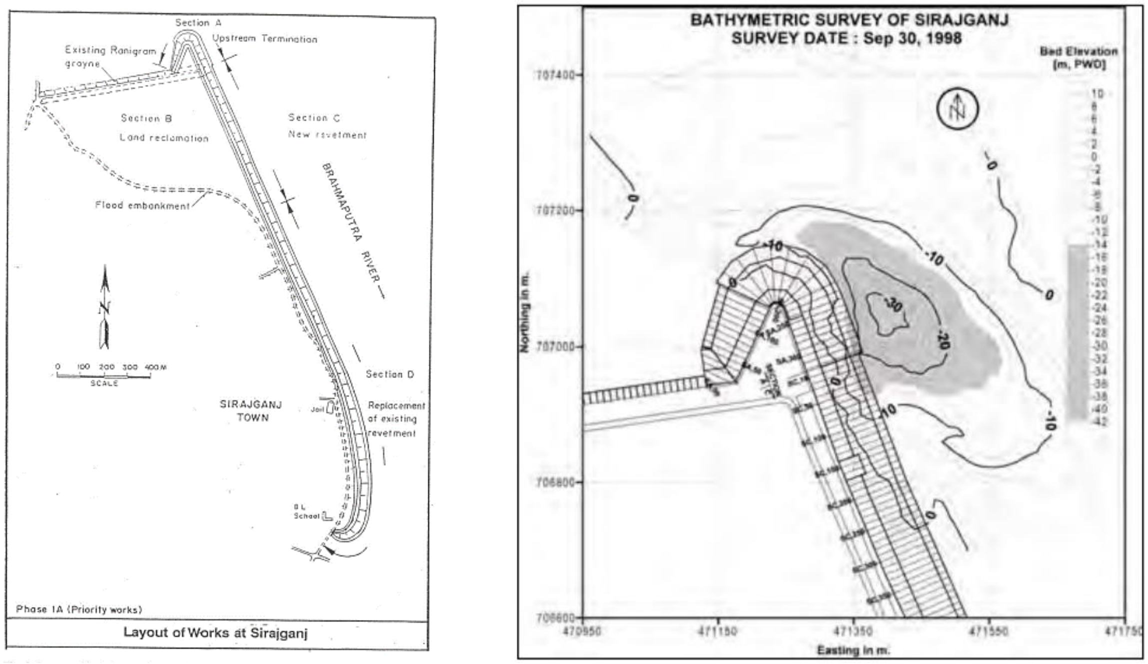


Figure 5 Final layout of training works for Sirajganj town protection and prototype measurements. Note the scour exceeds 30 m below PWD datum.

In 1998 serious damage occurred to the structure as a large flow slide occurred just south of the new upstream groyne-nose. Evaluations of hydraulic and geotech showed that the reason for the flow slide was the formation of the deep scour hole in combination with the loose soil that in this region has a quite high content of mica in the sand deposits. Mica is a mineral composed of plate-shaped grains.

8. SCOUR TESTS OF REVETMENT STRUCTURES

The Consultants further conducted series of model tests of scour in undistorted physical models with movable bed in scale 1:100. These tests were made in a 5.5 m wide flume with current set up for the purpose. The model sand was approx. 0.18 mm. Table 5 shows the results and calculations of the coefficient α in the Liu formula.

8.1 Vertical sided groyne

Fig 6 shows the measured scour and a photo of the model after testing. The vertical sided groynes gave values which are far above any other groyne tested. The initial water depth was 15 m and the scour hole 27.5 m giving a total water depth of 42.5 m. On this basis, an α value of 1.50 is recommended for design of such structures.

It appears that the scour hole for this type of structure always occurs very close to the corners and therefore the full scour depth would have to be considered for design purposes of the structure.

Table 5 Summary of BRTS model tests on local scour in undistorted model with live bed

Model no	Structure type	Maximum velocity near structure		Initial water depth, $\bar{y}$ (m)	Measured maximum scour, y_s (m)	Total water depth (m)	Scour at toe of structure (m)	Length of groyne (m)	Mean upstream velocity (m/s)	Froude number, F	$y_s/\bar{y}$	$L/\bar{y}$	$F^{0.33}$	$(L/\bar{y})^{0.4}$	α
		Prototype (m/s)	Model (m/s)												
13A	Vertical sided groyne	4.1	0.50	15	27.5	42.5	24	80	2.83	0.23	1.83	5.33	0.62	1.95	1.50
13B	Vertical sided groyne	4.5	0.55	15	28.5	43.5	27	80	2.86	0.24	1.90	5.33	0.62	1.95	1.57
14A	BRTS type revetment	5.6	0.56	20	9.0	29.0	7	145	3.56	0.25	0.45	7.25	0.63	2.21	0.32
14B	Groyne (slope 1:2) with apron	5.0	0.50	20	8.6	28.6	5	105	3.56	0.25	0.43	5.25	0.63	1.94	0.35
14C	Groyne (slope 1:2) no apron	5.3	0.53	20	13.6	33.6	6	105	3.56	0.25	0.68	5.25	0.63	1.94	0.56
15B	Groyne (slope 1:3.5) with apron	5.6	0.56	20	13.7	33.7	9	170	3.58	0.26	0.69	8.50	0.64	2.35	0.46
17	Groyne (slope 1:3.5)	6.0	0.60	20	20.5	40.5	18	152	3.60	0.26	1.03	7.60	0.64	2.25	0.72

$$\alpha = \frac{\left(\frac{y_s}{\bar{y}}\right)}{\left(\frac{L}{\bar{y}}\right)^{0.4} \cdot F^{0.33}}$$

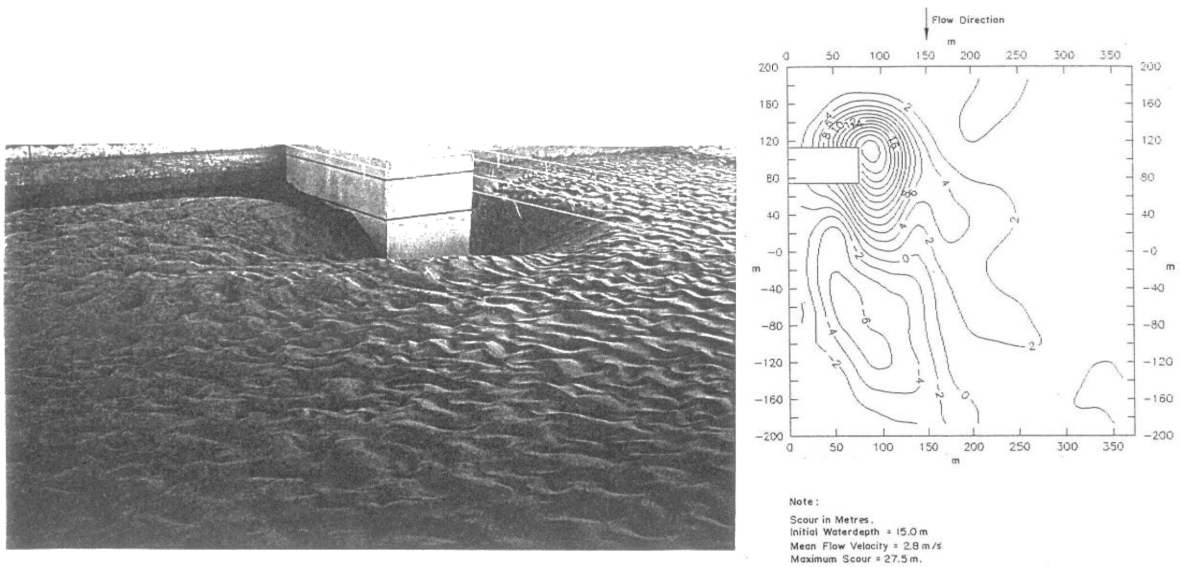


Figure 6 Photo from model tests of vertical sided groyne and measured scour after testing

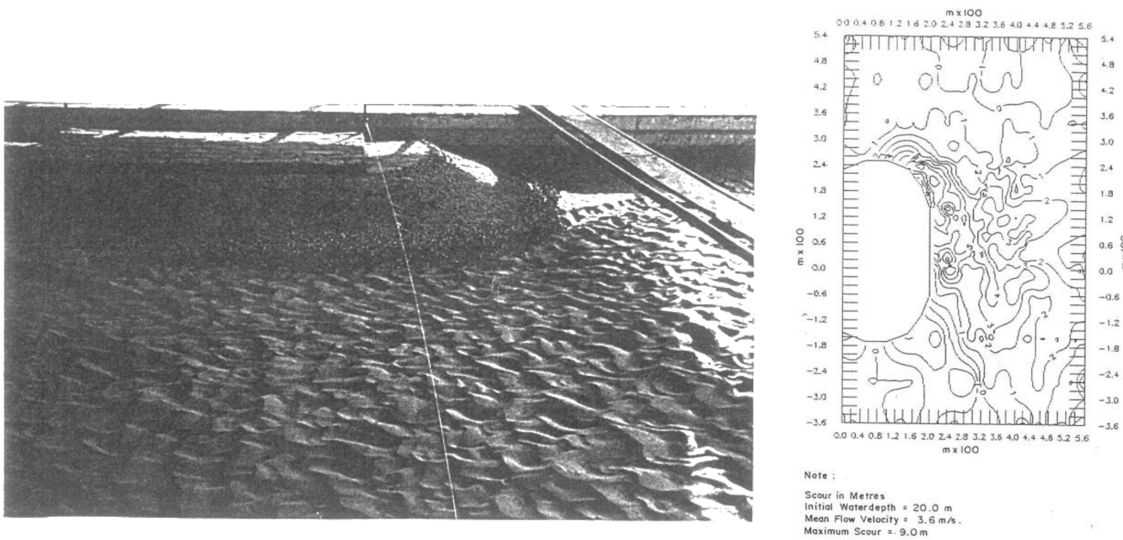


Figure 7 Model tests of BRTS type revetment (measured scour and photo after testing)

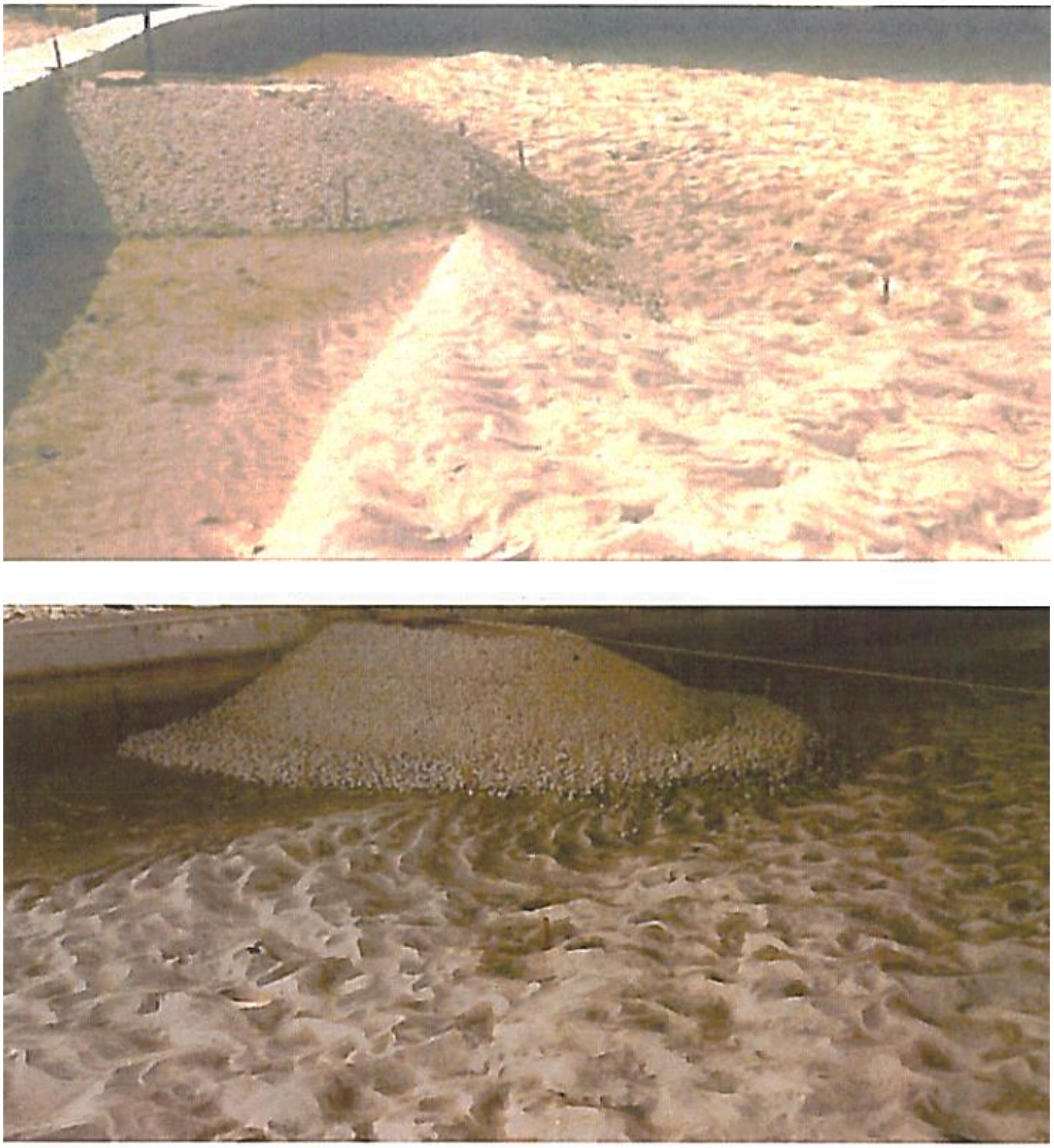


Figure 8 (1) Model tests of groyne, sloping structure with no apron. Scour 13.3 m (67% of depth)
(2) Model tests of groyne, sloping structure with toe apron. Scour 8.6 m (43% of depth)

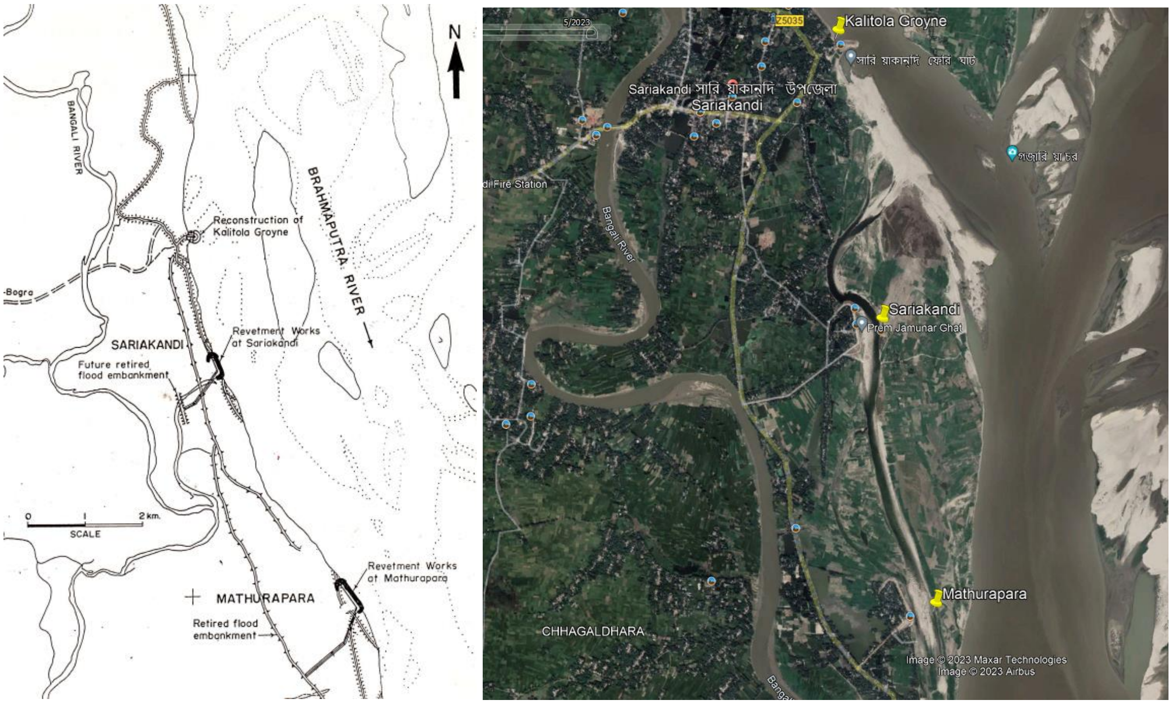


Figure 9 Location of Kalitola Groyne and Sariakandi and Mathurapara Hard Points.

In Figure 9, the left plan is from from 1992 and right Google Earth 2023. Note the southern hard point position was shifted nortwards for construction , and now the hard points are not under attack due to development of char land in front.

8.2 Single groynes with side slope

The test results (Figure 8) and prototype observations for groynes with side slope show some scatter. The coefficient, α, was found to be about 0.85 from the field observations, about 0.80 in the distorted model test for Ranigram Groyne, and 0.72 in undistorted model tests.

It is further evident that in most cases, the deep scour hole is located away from the structure. Only in Model 17 was the scour hole close to the toe of the structure. The maximum scour was 20.5 m and the maximum scour at the toe 18.0 m. The latter value of 18.0 m corresponds to a value of α of 0.63.

In the distorted model tests for Ranigram groyne, the α value close to the structure was as low as 0.30.

On this basis, it appeared reasonable as design guidelines to distinguish between maximum scour depth and maximum depth at the toe of the groyne. The recommended values for α (maximum depth) and α, (maximum scour) are selected to be, α = 0.90 and α =0 .65.

To account for migrating dunes, 2.0 m should be added to the maximum design depths.

8.3 BRTS type revetment

The BRTS type revetment (hard-point structure) was also tested both in the undistorted scour model in scale 1: 100, and in the revetment stability tests in scale 1:50. In the 1:100 scale test, the maximum scour was 9.0 m at the upstream termination and the total maximum water depth 20+9=29 m (see Figure 7 and the photo).

In Section 9, the revetment stability tests in scale 1:50 are described. The tests were performed in a 5.50 m wide customs-built flume at RRI. A certain blockage effect was present leading to higher flows near the structure than would otherwise be present. This was required to obtain a true similitude Froude model of the hard point structure in scale 1:50. In this model, the deepest scour hole (in the trough between dunes) was about 33 m relative to the l00y, WL.

In both cases, the scour hole was very close to the toe of the structure (within 10 m to 20 m).

The test in scale 1:50 was performed with deep water upstream of the revetment simulating a deep embayment. On this basis, the design scour depth of the BRTS type revetment was selected at 33 m for the upstream termination including a length of 200 m downstream. Along the trunk (straight section), the design water depth is 29 m. The studies provided the basis for the strengthening of Kalitola Groyne and design and construction of Sariakandi and Mathurapara Hard Points, Figure 9.

9. STABILITY TESTS OF BRTS HARD POINT STRUCTURES

Besides the model study to determine design scour for different types of groynes and hard points, a model investigation on the stability of the revetment structures in strong current was carried out. The study was made as an undistorted model in a purpose-built flume in a length scale of 1:50. The model set-up appears in Figure 10. The model had movable bed and the pre-test condition represents a situation with a general water depth of 20 m and 9 m scour reached the toe of the structure, i.e., the total maximum water depth is 29 m. The cubic blocks were 12 mm in model and the number of blocks used in the model was about 100,000. All depths refer to the 100-year water level. The model study was made in series and a total of 12 different models were tried each addressing a specific issue. Most of the tests concentrated on studying the stability of the armour on the slope, and especially the behaviour of the toe of the structure which was made as a so-called “falling apron” Different levels and width of the apron were tested as well as different material (cubic blocks or rock). It was discussed to use geotextile bags with sand for the falling apron, but this concept was not adopted. It is interesting, that this is now a days the preferred and adopted national preferred concept in Bangladesh/6/. It is selected due to that experience now shows that the bags are providing a better coverage of the underlying sand, and it is also a more economical solution.

In the model, the stability of the armour layer consisting of cubic blocks was investigated. The observations of the stability of the armour layer were compared with measurements of the local flow velocity over the surface of the structure. The following is a summary of the most important findings of general interest.

9.1 Armour layer

The stability of the armour layer blocks as function of the local flow velocity is shown in Figure *10* together with the adopted design formula for stability of armour blocks in current. The formula is denoted JMBA equation in Ref. /6/. It is the Izbash formula, that was later on updated to be more generally applicable by Pilarczyk. The results in Figure 10 also shows for a typical situation the velocity distribution near the structure, and a flow induced failure of the armour layer in the straight section just adjacent to the upstream termination.

9.2 Falling apron

Figure 11 focusses on the falling apron. It was quite remarkable the way the apron launched itself down the lower scoured sand slope developing a slope slightly flatter than 1:2. It was observed that at the beginning of a test, the blocks at the edge of the apron slowly displaced themselves down the sand slope because of local scour around the individual blocks due to local flow amplification. The launching of the blocks continued as a gradual process (see Figure 11) until the entire scoured slope was tightly covered with one layer of blocks and the space between the blocks was so small that the sediment on the slope was no longer moved by the current. The launching of the blocks thus resulted in a pavement consisting of one layer of blocks (see typical situation in Figure 11 for a profile with scour depth of 16.6 m below the apron).

Similar arrangement of blocks was observed in nature at Sirajganj, but the process of the launching of the blocks was only observed in the model. The process of the launching of the blocks is thus a relatively slow process to which the denomination "falling apron" does not apply. The process of self-armouring could rather be characterised as a sliding or "creeping apron". The tests further showed that the process of self-armouring occurred in the same way regardless of whether the apron material was cubes or rock (boulders). In a model, the sand is the same as in nature, so in nature a layer of large blocks or boulders cannot protect the sand from being sucked out through the gaps in the armour layer. Therefore, one would expect that over time, the amount of sand lost would be so large, that the structure would fail. One major challenge for construction of revetments with falling aprons is the fact, that it is safer if the falling apron can be constructed at a low level, which would mean that the height of the scoured slope that shall be covered with the falling apron after launching is reduced. To get the level of the apron low requires normally dredging of a very deep trench, and this is difficult because of the stability of the dredged slopes, and because the river flow will quickly fill the trench up again. This is also why today for RTW structures in Bangladesh an adaptive approach has been adopted, where it may be required to come back and replenish the material in the falling apron after launching. There are examples where this has been done in practise after the apron has fully launched, /6/. More recent experience on falling aprons is given in /12/ and /13/ including experience from nature and results from model tests. These tests with rock and geobags show the same as reported above for the BRTS tests with the apron material coming down the scoured slope in one layer until the whole slope is covered. The model tests are in scale 1:20. In general it would be advantageous that model tests are made in as large scale as possible, but getting to amore ideal scale of, say 1:10 to 15 is normally not possible within the available model facilities that would have to be very big and have a water system with a high discharge capacity. Also, many clients refrain from doing such more expensive test programmes although a specific project can benefit from it, and if results are published in public domain, the profession may gain important knowledge.

However, much smaller RTW projects can and should be used as "models" for the very large ones, as they would be in similitude and could be interpreted as models of the very large structures and general knowledge and understanding extracted.

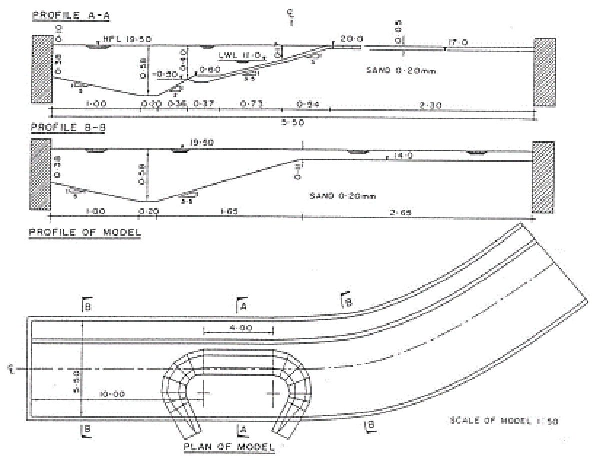
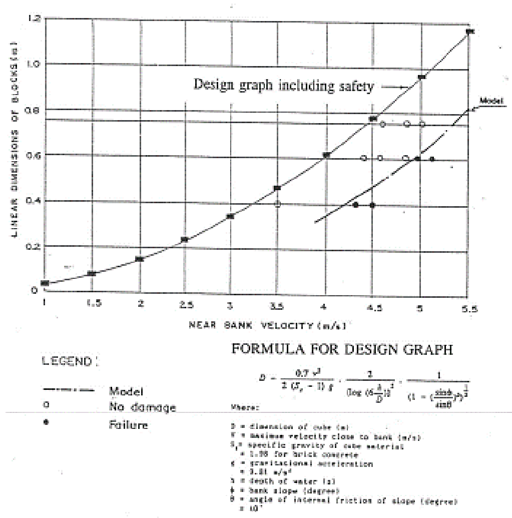


Fig 9 Model set-up for stability tests of revetment structures



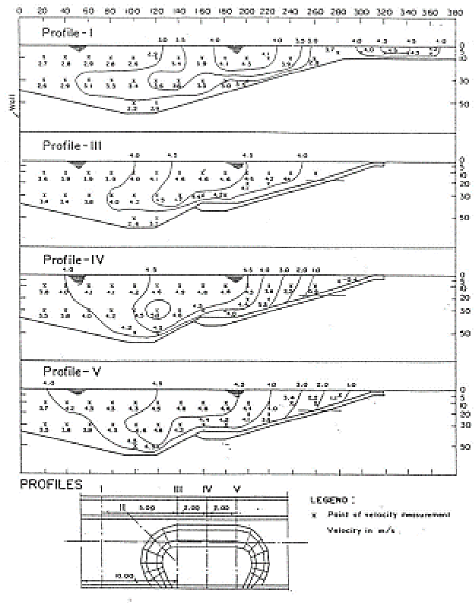


Fig 11 Typical velocity distribution near the structure

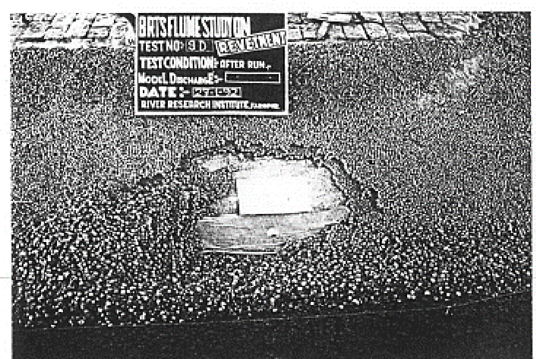


Figure 10 Model set up. Results of stability tests and photos during tests and after with damage

Another important finding of the model study was that heavier armour is required near the upstream section than on the rest of the structure.

Another important finding is the fact that the flow velocity is reduced near SWL and consequently the final design (see Figure 12) comprised three clearly different structural elements: i) The horizontal apron placed in an appropriate low level that is feasible from a construction point of view. ii) The lower slope below LWL +2.0 m. Here, the armour consists of two layers of randomly placed blocks constructed in the same way as for breakwater armour layers. iii) The upper part of the slope which is less exposed to current forces, and which can be constructed in the dry under close supervision. The armour consisted of one layer of concrete cubes. Calculations have shown that in a very large river as Brahmaputra, the largest environmental force on the upper part of this structural element is due to waves rather than current. It is important to note that below both the lower and upper slope, a geotextile is placed. In fact, the geotextile is the most important structural element, and the blocks placed on top are to be regarded as ballast serving the purpose of keeping the geotextile in close contact with the below-lying soil. The slope of the structure is 1:3.5. This relatively flat slope is for geotech reasons to ensure seismic stability of the slope consisting of the natural fine sand, d =0.18 mm.

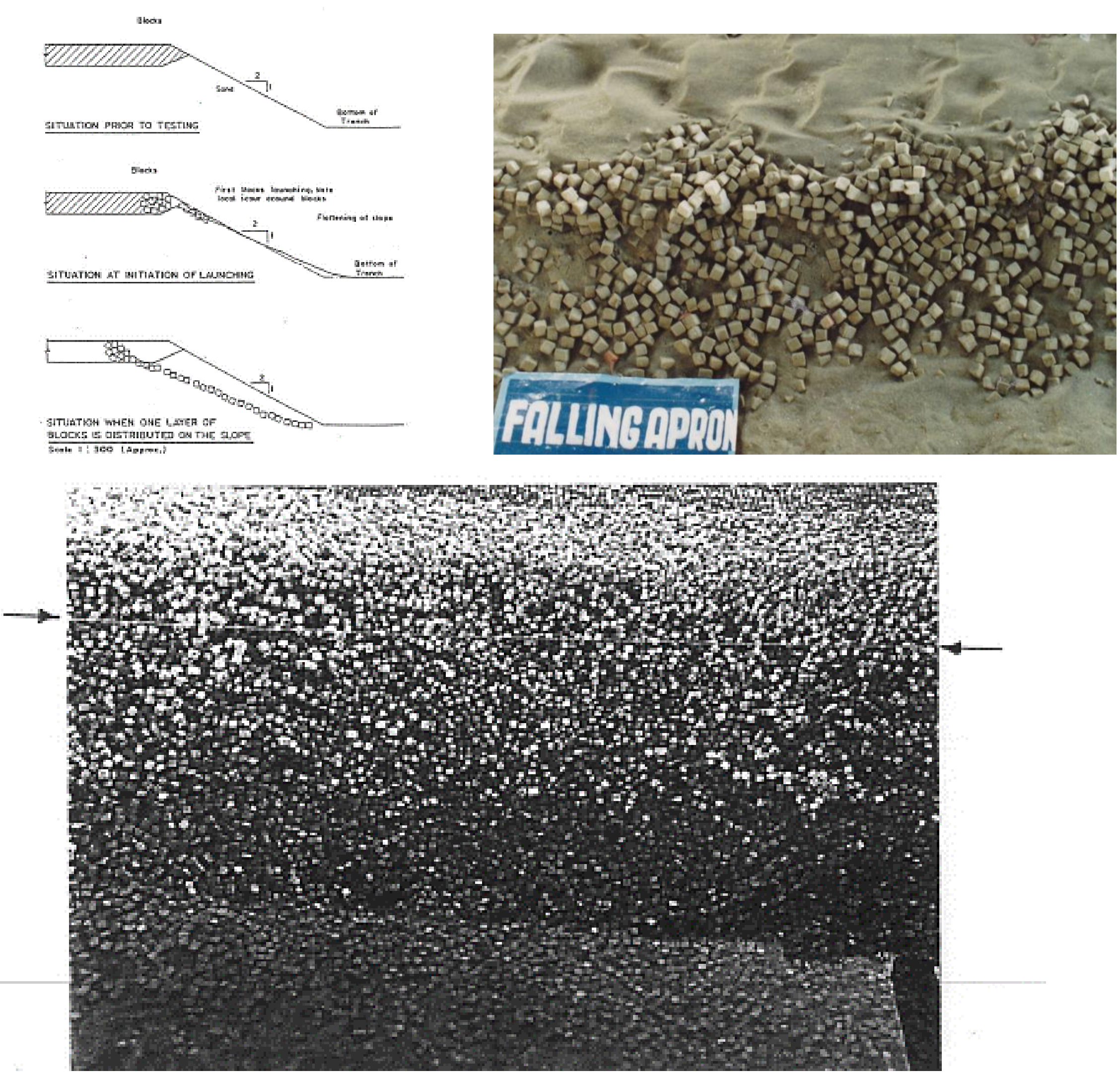


Figure 11 Tests with falling apron of 60 cm cubic blocks. Note development. Arrows in left photo indicate level of apron prior to launching. Note very smooth surface of blocks in one layer on the scoured slope develops. Photo to the right shows an apron in middle stage during launching.

The two hard points were constructed in the 1990ies and it interesting that the main worry at the time of the design, that the Brahmaputra should move further westwards to join with the Bengali river never materialized, and only limited scour appears to have occurred. Today 2023, large char-land has developed in front of the hard points. However, one never knows what will happen and the river may again sometimes in the future develop a large and deep flow channel at these hard points.

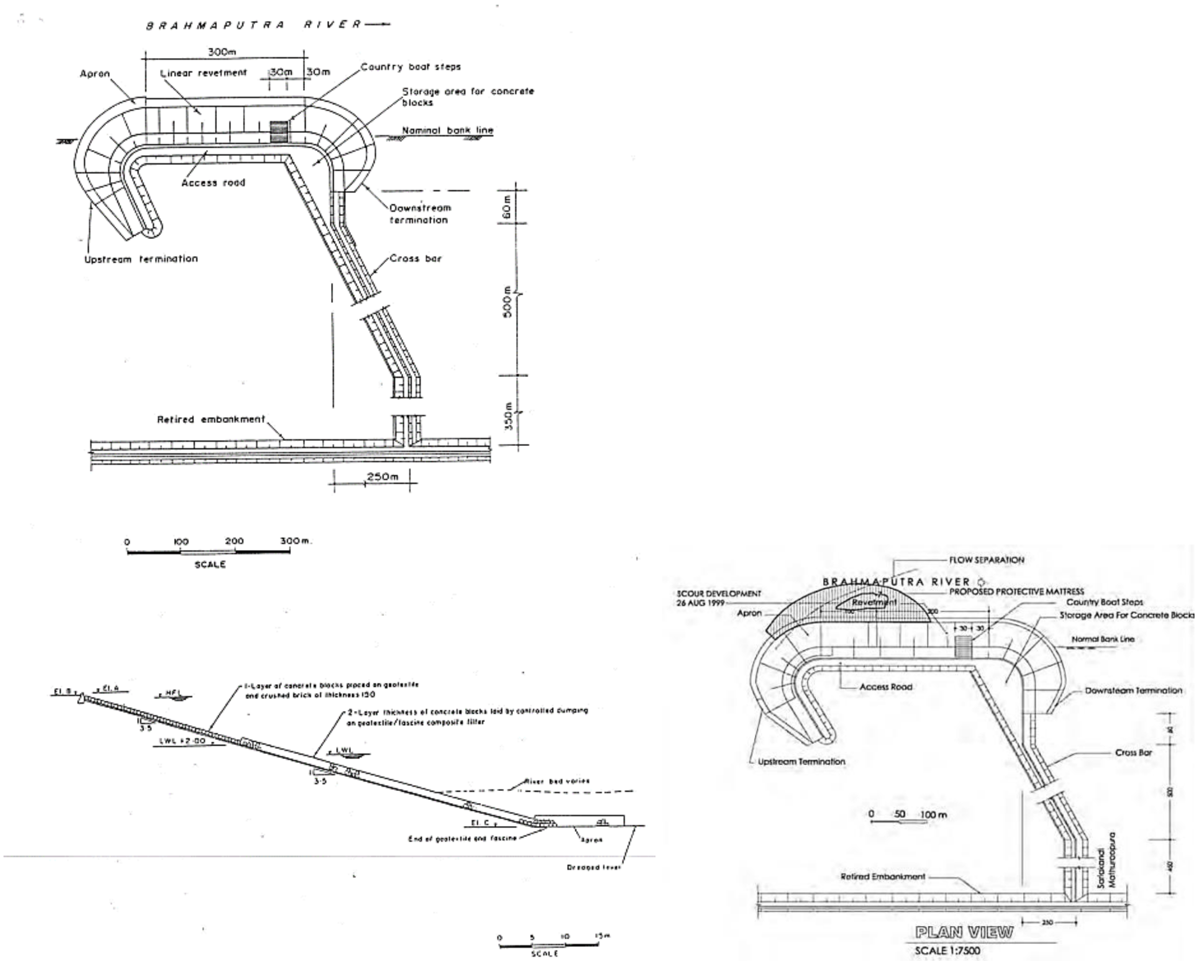


Figure 12 Design of hard point at Sariakandi and observation of scour development

10. COMPARISON TO JAMUNA BRIDGE SCOUR AND RTW STRUCTURES

At the time of the BRTS study in the early 1990ies, the Jamuna Multipurpose Bridge Project (JMBP) design was underway, and the huge size and importance of the study made it a proper reference to analyse and to gain knowledge from and to compare with the BRTS study results.

Figure 4: Revised layout of Jamuna Bridge guide bunds as built

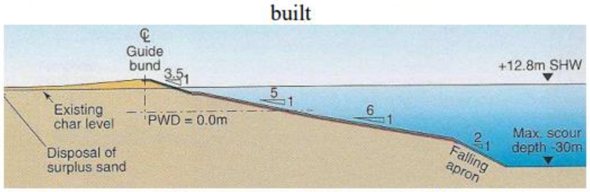
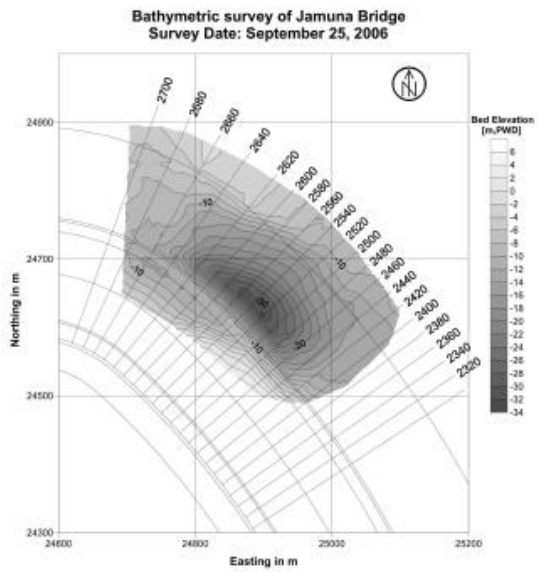


Figure 5: Typical cross-section Jamuna Bridge RTW (after launching of falling apron)



(b) 25-9-2006 stage PWD + 11.53 m, deepest scour PWD -33 m

Figure 12 Layout of the Jamuna Bridge.

Note, Figure 12 from /8/ shows how the river is squeezed to a width of ~4.6 km, and cross section of the guide bunds with design scour depth of -30 m and design WL of +12.8 m. To the right bathymetric survey Sept. 25, 2006, with deepest scour depth at -33m PWD for a water level of +11.5 PWD.

A comparison was made between the design velocities and scour depths of BRTS and JMBP. It was found in 1992 that the basic design velocity for the guide bunds at JMBP was only 3.1 m/s, but increased to design velocities of 3.4, 3.9 and 4.2 m/s in levels -15, -25 and -30 m PWD respectively; whereas the BRTS had 3.7 and 4.4 m/s (see Table 3) as direct design for straight and termination sections. Using the BRTS design velocities would have resulted in slightly larger rocks. The rock size was in 1990 reported as D_{50}=0.23 m, whereas it in /8/ is reported as 0.25-0.30 m. The paper /8/ reports in 2009 that the structure has in general been stable, except for a small section where the rocks have slid down on the bamboo mattresses, see Figure 13. The design scour level was originally set at -30 m, which with the design water level of +12.8 m means a total depth of ~43 m. It is interesting to notice that this is the same depth as predicted by the Sirajganj model tests (see Figure 4) and the actually maximum measured water depths at Sirajganj in 1986. Note, that /8/ is reporting scour to -33m PWD in 2001 and 2003, and states, quotation: *"It appears that design of bank protection works on the Bramaputra / Jamuna River should assume that up to 35 m of vertical scour at protruding structures and 15 m at parallel revetments can occur in the course of one flood season "*. This appear to be in general agreement with the BRTS studies and design assumptions. Figure 13 shows the construction of the guide bunds with the bamboo mattresses.

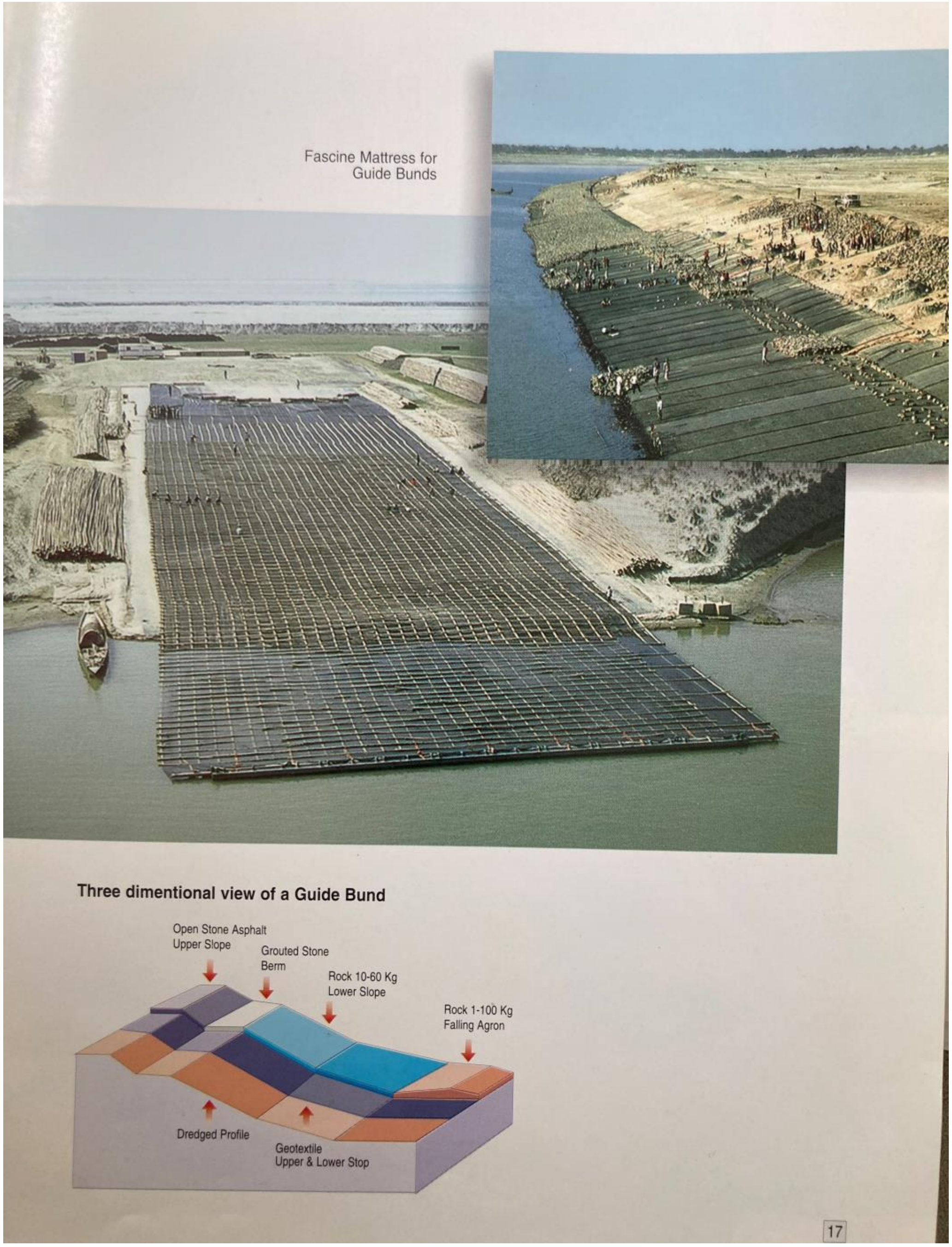


Figure 13 from /11/. Construction of the guide bunds using very large bamboo mattresses (level +10 to -15 m PWD) that are covered with a 10 cm thin layer of rocks 5-10 cm and two layers of armour rocks D_{50} =25-30 cm (10-60 kg).

11. CONCLUSIONS

The paper presents the results of the very important studies and design for securing the right (west) bank of the Brahmaputra River in Bangladesh. These studies led to the construction of a strengthening of an existing groyne at Kalitola and the design and construction of two large hard points at Sariakandi and Mathurapari, as well as construction of a new upstream groyne nose at Sirajganj town. First a design basis had to be developed based upon analysis of data of the river with respect to natural water depths and flow velocities. It was on this basis of importance, that the data allowed for a statistical study of maximum water depths in a 200 km stretch of the river, and it was found that the 1 % probability of occurrence natural depth as high as 28 m. The studies included also numerical modelling and many physical model tests of various types and configurations of RTW structures both with respect to scour, but also stability of the hard points. Further in the time since the construction of these projects, experience has been gained from field surveys that have been reported in public domain papers and publications. In general, it appears that the very large scour depths that was found in the model tests have developed in Sirajganj, while for the two hard point structures, it is a surprise that the morphology development has been in favour for the structures, as the river has shifted its course and at present there are char islands in front. From Goggle Earth these appear for the time being to be so stable that crops are grown on the islands. The situation at Sirajganj is different and the new structure termination has been under severe attach and a very deep scour hole more than 30 m deep has developed. With the actual water level this means a total depth of approx. 45 m at the structure nose, a depth that is in agreement with what was predicted based upon the model tests. Note, that results for the Jamuna Multipurpose Bridge (JMB) is also presented for comparison and here the maximum scour measured is also more than 30 m, which also here means development of a total water depth of about 45 m.Such extreme scour development at large man-made structures represents a severe challenge for design and construction of RTWs, and it appears from the reporting of practical experience, that the national strategy procedures have now changed. It now allows for adoptive designs, where it will be required to come back and replenish or otherwise strengthen a structure when required once the falling apron has launched due to formation of a deep scour hole. With respect to construction materials, it is to be noted, that now a days the preferred material for a falling apron is sand filled geotextile bags, where traditionally it was cubic concrete blocks. It is the hope that the experience presented in this paper can be of more general value for designers of large RTWs in large alluvial rivers also in other countries than Bangladesh.

12. ACKNOWLEDGEMENTS

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