COMPARATIVE ANALYSIS OF DESIGN AND PERFORMANCE OF BANK PROTECTION WORKS OF JAMUNA RIVER AT TITPOROL AND DEBDANGA.

Submitted by

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In partial fulfillment of the requirements for the degree of Master of Engineering (In Water Resources Engineering)

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We here by recommend that the project prepared by MD. ANISUR RAHMAN entitled "COMPARATIVE ANALYSIS OF DESIGN AND PERFORMANCE OF BANK PROTECTION WORKS OF JAMUNA RIVER AT TITPOROL AND DEBDANGA." be accepted as fulfilling this part of the requirements for degree of Master of Engineering in Water Resources Engineering.

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CERTIFICATE

This is to certify that this project work has been done by me and neither this project nor any part thereof has been submitted elsewhere for the award of any of my degree or diploma.

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ABSTRACT

Bangladesh is located in the lower part of the delta having flat land formed mainly by the sediments deposited by the world's three largest rivers (GBM), the Ganges, the Brahmaputra and the Meghna. Huge quantity of flow during monsoon coming from the upper catchments (located outside the country) makes its way towards the sea through these rivers. Due to the geographical location of Bangladesh on the globe, the river systems here are morphologically very much active which is evident from the continuous wide spread bank erosion, channel shifting and sedimentation processes. Bangladesh, on average, is losing more than 8,000 ha of land annually due to bank erosion. Although there is a tendency of decreasing rate of erosion due to implementation of large number of riverbank protective works during the last few decades by Bangladesh Water Development Board (BWDB). Failure of implemented bank protective works is also remarkable. Under these circumstances, this study has been conducted to determine the probable cause of the damage of riverbank protective works constructed on the right bank of Jamuna River at some selected locations of Sariakandi.

BWDB implemented the protective works at Titporol and Debdanga along the Right Bank of the Jamuna River during November 2004 to April 2005. From field investigation on 28 June 2005 it was found that the revetment work at Debdanga performed well during early flood after construction. But some portion of upstream revetment at Titporol, however, damaged in June 2005 and mitigation measures by dumping of sand filled synthetic bags were carried out to stop further collapsing of river bank.

In order to find the probable causes of failure of protective works at Titporol, investigation has been carried out through checking the adequacy of the design of the revetment, slope stability analysis and field investigation.

Design of the revetment has been reviewed using the standard procedure mentioned in the Design Manual of BWDB and have been found satisfactory. From the analysis of field condition it is revealed that the damage of revetment works occurred due to low shear strength of soil and the presence of pore water pressure developed behind the geo-textile filter. It is apprehended that for the lack of free drainage, pore water pressure developed behind the geo-textile filter and this resulted failure of bank slope. At the damaged portion, subsoil water might have been drained from underground source or from the existing ponds behind the revetment works as was found at the protective site of Titporol. On the other hand, there found no such kind of underground source of water or existing ponds near the revetment- works site at Debdanga. Therefore, the protective-works at Debdanga performed well.

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ACRONYMS AND ABBREVIATIONS

ASCE	**American Societies of Civil Engineers**
ASL	**Anticipated Scour Level**
BRE	**Brahmaputra Right Embankment**
BWDB	**Bangladesh Water Development Board**
DWL	**Design Water Level**
FAP	**Flood Action Plan**
GBM	**Ganges Brahmaputra and Meghna**
HWL	**Highest Water Level**
IWM	**Institute of Water Modelling**
JMREMP	**Jamuna Meghna River Erosion Mitigation Project**
LWL	**Lowest Water Level**
O&M	**Operation and Maintenance**
RBPP	**River Bank Protection Project**
RWL	**River Water Level**
PWD	**Public Works Department**

CHAPTER ONE

1. INTRODUCTION

1.1 General

Bangladesh is located in the lower part of the delta where the world's three largest rivers (GBM); the Ganges, the Brahmaputra and the Meghna meet with the sea. These three rivers drain a total catchments area of about 1.72 square kilometers in India, Nepal, China, Bhutan and Bangladesh. More than 90% of these catchments are located out side the country and huge quantity of flow makes its way towards the sea through these river systems. Due to this geographical location on globe, the rivers of Bangladesh are morphologically very active which is evident from the continuous wide spread bank erosion, channel shifting and sedimentation processes. River bank erosion problem here in Bangladesh is a regular phenomenon as a result of which the country is constantly facing immense multi-dimensional socio-economic problems. Bangladesh, on average, is losing more than 8,000 ha of land annually due to bank erosion. Although different study reports confirm that there is a tendency of decreasing rate of erosion due to implementation of large number of riverbank protective works during the last few decades by Bangladesh Water Development Board (BWDB). Failure of implemented bank protective works is also remarkable.

Along the right bank of Jamuna river about 5.6 km area between the Hasnapara Spur 2 and Kalitola Groyne (near Titporol) was opened to a major fraction of the Jamuna flow and at this area the right branch of Jamuna was flowing obliquely to the bank. Due to this oblique flow during the 2003 monsoon, about 187 m of area was washed away and that time the Brahmaputra Right Embankment (BRE) was only about 40 m away from the Jamuna River. If bank erosion would continue at this rate, then there would a threat of unification of Bengali and Jamuna flow at this area. This probable unification of Bengali and Jamuna River is not only a threat to the adjacent area of Bogra district, but also is a threat to the existence of the Jamuna Multipurpose Bridge. Under these circumstances, Bangladesh Water Development Board has under taken a project entitled "Prevention of Merger of the Bengali and the Jamuna River at Titporol and Debdanga under Sariakandi Upazila in Bogra district". Based on mathematical study carried out by IWM, Bangladesh Water development board implemented the riverbank protective works at

Titporol and Debdanga in 2005. Under the same hydrological condition both the protective works have been implemented. The work at Titporol has shown several slope failures after implementation in 2005. But the protective work at Debdanga is functioning well without any damage. Considering the importance of sustainable bank protective works in Bangladesh, it is necessary to find out the causes of bank failure. Under this circumstance, the study has been considered in the proposed research work.

1.2 Objective of the study

The main objectives of the study are :

i. To assess the adequacy of design of protective works at selected sites (Titporol and Debdanga)
ii. To compare the performance of protective works at Titporol and Debdanga
iii. To investigate the causes of failure of protective works at Titporol.
iv. To suggest possible remedial measures for sustainable slope protection

1.3 Organization of the Report

This report describes the overall approach and methodology followed in the study and the findings. The report contains one report with three appendices, Appendix-A: Sample Design Computations, Appendix-B: Slope Stability Computation and Apendix-C : Flood frequency Analysis.

Chapter 1 describes the background, the objectives of the study, Chapter 2 describes review of literature, Chapter 3 represents study approach and methodology, Chapter 4 presents the hydraulic and morphological characteristics of Jamuna river, Chapter 5 presents the performance analysis and investigation of the causes of failure of revetment and Chapter 6 describes discussion and conclusions.

CHAPTER TWO

2. REVIEW OF LITERATURE

2.1 General

River bank erosion is a major problem in Bangladesh. Bank erosion occurs in stable as well as unstable river channels. The need of river training and bank protection in Bangladesh arises from the fact that most rivers of the country are unstable, i.e. they are not in a state of equilibrium with the governing physical processes. Unstable river undergo permanent and rapid changes of the water and sediment regime and, hence, adjust in depth, slope, width and plan form accordingly. Both, river training and bank protection measures, which are strongly interrelated, have the objective to ensure a safe and efficient transport of water and sediments (suspended material and bed load) through a certain defined stretch of the river

2.2 Erosion and Scour

Erosion : Generally the removal of the bed material from the land surface is called the erosion. But when the term is related to the bank revetment works, it may be defined as the removal of the bed materials and the sediment particles from the river bed causing reshaped of the particular places. Someone has defined erosion as the mechanism of the detachment of the sediment particles and other materials from the land surface. There is another definition which defines erosion as the combination of the processes of the detachment and the transportation of soil particles and other materials from the bank and bed of the rivers. (ASCE, 1970). Thus, we can say," Erosion is the loosening or dissolving and removal of the earth or rock materials from any part of the earth's surface". However, the bank erosion may be defined as detachment or the dislodgement of the soil particles from the bank by the action of flow, waves, tidal fluctuations and other hydrological factors governing the condition of a channel.

Scour : Scour may be defined as the removal of the soil particles from the soil water interface by current or wave induced share forces within the waterway; possibly in combination with hydraulic gradient forces or by rainfall run-off above the water line. It may also be defined as the abrupt decrease in the river bed elevation of channel due to erosion of the bed materials by the flow. The local drop or depression found in bed of a river or channel found in the region of a structure is known as the local scour. This happens due to the interaction between the velocity of flow and the bed materials causing them weak and as a result there is a modification in the pattern. The knowledge of the maximum depth of scour around hydraulic structures is essential from the point of view of safety of the structures, excessive scour can undermine the foundations and lead to the failure of the structures.

According to the experiments carried out by Rouse in 1940, it has been found that (a) the depth of scour Ds in a uniform material is dependent solely upon the thickness (T) of the sheet of falling water, velocity of the jet V, the fall velocity of the sediment particles W, and the duration of the scouring action; (b) the relative rate of scour produced by a given jet Ds/T at a given stage (time) depends only upon the ratio of jet velocity to fall velocity V/w (UN-ECAFE, 1953).

An empirical formula for maximum scour at noses of guide banks as given by Inglis and Joglekar is, $Ds = 1.3 (Q/F)1/3$ 2.1

where Ds is the depth of scour in feet, Q is the maximum discharge in cubic feet per second, and F is the Lacey silt factor. In addition to the above formula, Inglis and Joglekar gave the following formulae :

Scour at straight groynes, facing upstream, with steep sloping head (1.5 to 1.0).

$$Ds = 1.8 (Q/F)^{1/3} \quad (2.2)$$

and with long sloping heads (1 in 20)

$$Ds = 1.3 (Q/F)^{1/3} \quad (2.3)$$

For other groynes along the river bends,

$$Ds =(0.81 \text{ to } 1.8)x(Q/F)^{1/3} \quad (2.4)$$

Hossain (1981) also carried out laboratory experiments on maximum scour depth at the nose of solid groynes. He arrived at the following equations :

For single groyne, $ds = 0.205 (b/w) + 0.196$ (2.5)

For double groyne, $ds = 0.583 (b/w) + 0.046$ (2.6)

where b/w is the groyne projection ratio and ds is the maximum scour depth.

2.3 Types of Scour

Normally there are two type of scour, such as natural scour or general scour and local scour.

Natural scour or General scour – it occurs at the following conditions:

1. Scour at bends/Bend scour. This type of scour happens due to helical flow patterns associated with bend. The maximum scour occurs on the outside of a bend somewhat downstream of its apex.

2. Scour at Confluences/Confluences scour: This type of scour is visible immediately downstream of the confluence of two anabranches. This happens due to the combination of converging streamlines and finally causing the turbulent flow.

Local scour – it is structure-induced scour and usually happens due to the concentration of flow lines in the vicinity of structure combined with local shear induced turbulence caused by the roughness of the structure.

2.4 Process of Erosion

The erosion of any place may take place in the following two ways:

1. The displacement of the solid particles from adhesive / cohesive contact when taken place causes the erosion.

2. The transportation / washing away of the solid particles from the particular position/ site/place taken place causing the underlying materials attack or wash and finally there happens an erosion. According to the California Highway Practice, 1970, this type of erosion is nothing but a natural consequence of the flow passing through a solid boundary (California Highway Practice, 1970).
   The resistive force developed along the contact surface of a moving body with a stationary body is called friction. When the two bodies are solids, friction is essentially a function' of the texture of the surface in contact and the pressure between them, but when one of the bodies is a fluid, the conflict along its bounary disturbs its motion for some distance away. The zone in which this disturbance is significant is called the turbulent boundary layer, The thickness

and dynamic character of this layer depend on roughness of the solid boundary and velocity of the passing fluid and is independent of pressure.

The solid particles on the river bed or on the bank of the river become exposed due to the flow of water or fluid within very short period of time due to the following reasons:

1. Water/fluid flowing tangentially towards upstream;
2. Water/fluid flowing tangentially towards downstream;
3. Direct impinging and
4. Direct retreating together with intermediate combination.

Due to any of the reasons mentioned above, a loosely bonded particle of the bank or bed materials are projected from upper layers of the bank or bed. The impinging particles of water tend to take part in vibration and loosen the solid particles. The retreating particles introduce sucking called cavitations which is very much harmful and destructive. Therefore we may conclude that erosion of particular location of the river bed or bank is related to the roughness of the river bed or bank and the velocity of the flowing fluid of the channel. Here to mention that, more the roughness more the erosion and more the smoothness less the erosion. That is why a smooth bank or bed does not go under more erosion by the tangential flow at even tremendous velocity. River bank is to be classified as an ideal smooth bank when it will be able to endure fairly high velocity. But when the roughness of the bank or bed becomes significant, the tolerable velocity decreases. No doubt the erosion will take place more or less that depends on the bonding forces of the particles forming the bed or bank materials. The cohesive materials of the bank or bed go under less erosion and vice versa. Vegetables on the bank or bed play an important role on the decease of river erosion.

2.5 Theory of Erosion and Scour

When solid particles from the river bed or river bank are washed away or removed from the particular places causing the depression in elevation with respect to the surrounding places are called scour and erosion. This happens when the soil particles composing together from the particular places are acted upon by the forces sufficient to cause them to move. Here we may give an example for a straight channel of uniform depth. If • is the unit weight of water, R is the hydraulic radius and S is the slope of the river, then according to the tractive forces theory, the average tractate force / the acting force of running water on the river bed is equal to •RS. If the river be is non-cohesive, there is a definite relation between the critical tractive force and the mean diameter and specific gravity of the river bed material. Reference may be made to Table 2.1 wherein the critical tractive force for different sizes of bed material is given.

For a meandering river, the actual tractive force will be larger than the mean value given by the expression RS on account of the shifting of maximum velocity towards the concave banks as well as the development of secondary currents (UN-ECAFE, 1953). The percentage of increase of tractive force, or the decrease of river bed resistance as compared with a straight channel, suggested by Lane (1951) for different degree of sinuosity of channel (is given in Table 2.2).

The statement made above refers to the erosion of the non-cohesive bed of rivers. On a slope, as pointed out by Fan (1947), the resistance of a sediment particle to motion is reduced by the sliding force of the particle itself on an inclined plane due to its own weight. If K is the ratio of tractive force required to move a particle on the inclined plane to that required to move the same particle on a level bottom, Fan (1947) shows that for non-cohesive materials K is affected by the angle of repose ø of the material and the angle of side slope • and can be expressed by the following equation :

$$\mathbf{K} = \sqrt{1 - \frac{\sin^2 ø}{\sin^2 \theta}}$$

Another important factor to be considered is the distribution of tractive force on the side slope of river bank which has been worked out by olsen and Florey (1952). The tractive force is a place somewhere between o.2D to 0.3D above the bottom and decreases to zero at water surface wher D is the depth of water.

The above derivation is for non-cohesive material only. If the material is cohesive, size of particles may be reduced. For irregular and turbulent flow, such as that below a dam or weir, or wave wash, the actual force may be much greater than the tractive force thus computed.

The tractive force which when becomes just sufficient to set the bed materials of a certain kind into motion is called the critical tractive force. There is a particular relationship between the mean diameter plus the specific gravity of the particles of the river bed material and the critical tractive force. The velocity of flow is automatically shifted towards the concave banks due to the development of the secondary current in the meandering section of any river. For this reason, the actual tractive force will be greater than the average value of the tractive force which is obtained by the formula •RS.

The above mentioned theory, description, or statement / remarks etc. is only applicable for particles of non-cohesive materials of the river bed. In 1947 Fan mentioned that the resistance of a sediment particle to motion is decreased by the sliding force of the particle itself on the sloping surface or on an inclined plane due to its self weight. Fan also mentioned that the ratio of the tractive force required for the motion or movement of the particles on the inclined plane to the tractive force required for the movement of the particles on a level surface(level bottom) for non –cohesive materials is affected by the angle of repose of the material and angle of the side slope. Olsen and Florey have pointed an important factor on the distribution of tractive force on the side slope of the river bank. According to them the tractive force is zero at the bottom of the side slope increase to a maximum at a place some where between 0.2d to 0.3d above the bottom and decrease to zero at water surface where d is the depth of water. Here to mention that there are cohesive and non –cohesive materials which may be available in the formation of river bed. But the above mentioned theory, formula and derivation are only applicable for non-cohesive materials. The shape and size of the particles of the bed and bank materials may be reduced if they are cohesive in nature. The flow below a dam/ embankment/ sluice/ or the flow over the wire is irregular type of flow. For such kind of irregular or turbulent flow the tractive forces which are calculated from the above mentioned theory / formula becomes much less than the actual forces developed there.

2.6 Causes of Bank Erosion

A bank may fail owing to any one or a combination of the following reasons

1. Washing away of the soil particle of the bank by current or by the waves which is called erosion.

2. Sliding due to the increase of the slope of the bank as a result of erosion and scour.
3. Undermining of the toe of lower bank by current, wave, swirls or eddies followed by collapse of overhanging materials deprived of support which is called scour.
4. Sloughing or sliding of the slope when saturated with water, this is usually the case during flood of long duration.
5. Sliding due to seepage of water flowing through bank into the river after receding of the flood, the internal shearing strength is considerably decreased owing to saturation and the stability is further decreased by the pressure of the seepage flow.
6. Piping in the sub layer due to movement of ground water to the river which carries away sufficient material with it.
7. Scouring of bed and bank by eddies with horizontal axis when flow occurs over a reef or submerged structure

The various hydraulic actions responsible for the erosion in particular location can usually be classified as follows (After California Highway Practice, 1960)

1. Frictional erosion by tangential flow.
2. Impingement erosion by curvilinear flow.
3. Eddy erosion below restriction.
4. Kolk scour below reef.
5. Erosion in varied flow
6. Erosion in unsteady flow
7. Wave action
8. Static erosion

2.7 Methods of Bank Erosion Protection Measures

2.7.1 General

In order to prevent or minimize the loss of valuable land, several stretches of the river banks might need suitable protection against erosion. Design of protection structures requires control of erosion by installing properly designed filters covered usually by large sized materials (stones, concrete blocks etc.) to absorb the energy of moving water.

Due to non accountability of different field circumstances and construction limitations experienced in field protection structures fail to perform satisfactorily in many occasions. Reconstruction or repairing works of the same structures creates huge economic burden for the country.

At present, geo-bags are used in bank protection works as stable protective elements. In Bangladesh, geo-textile materials were employed on a larger scale since 1994. Commonly sand filled geo-bags are used in emergencies but not for long lasting and large scale bank protection. To develop low cost alternatives for riverbank protection, Bangladesh Water Development Board took up an experimental river erosion mitigation project named Jamuna Meghna River Erosion Mitigation Project (JMREMP). The project looks back on eight years of implementing effective and low cost river training at several places of the Jamuna and Meghna River. Sand filled geo-bags were identified as a low cost effective solution during a pre- feasibility study in 2000 and subsequently they were used by BWDB in 2001. However, very limited investigations are carried out to evaluate the performance of geo-bags as a protective measure for rivers in Bangladesh. Therefore, engineers should gather knowledge regarding the performance of geo-bags as a bank and bed protection materials when applied to different field conditions.

2.7.2 Bank protection method

A river bank is nothing but a sloping surface of the adjacent land coming into contact with the surface of the still or flowing water. Protection means to save something from the attack of someone. Therefore bank protection means to save the river from the action of wave or current of the river water. It includes all type of protective works having the target of maintaining the stability of the land against the action of flow of the river water or surface runoff. This definition includes all types of protection works such as protection works of shore lines done along the sea shore and lakes against waves action and drift, protection work along the navigation canals against wave action generated due to passing of the various type of vessel and the protective work along the embankment and banks of the rivers for the purpose of the flood control.

A river bank may be divided into three parts, such as 1. Embankment portion which is called the sloping surface of the embankment facing the river. 2. Upper bank – that part of river bank which is located above the low water level and below the shore level. 3. Lower bank which portion of the river bank is in below of water surface. Depending on the different parts of the river bank different methods of protection measures are taken.

2.7.3 Classification of bank protection measures

Type of protection measures

Type of works to be done for the purpose of training a river or a reach there of is dependent on the objective and engineering principle chosen to be adopted in the river training program. The usually adopted type of river training works are described below with specially reference to their important features and applicability.

Bank revetment

Revetment is a bank protection measure where the bank slope is covered with erosion-resistant materials. The function of the revetment is thus to reduce the hydraulic load acting on the soil and possibly to facilitate stabilization of the soil. This type of river training works involve a protective cover of a suitable hard material applied on the slope and toe of the river bank so that the bank soil protected from the actions of erosions of erosive forces of flowing water and dynamic actions of waves. Revetments are of the two types:

1. Open joint type in which joint gaps between individual hard materials remains open allowing free flow of water.

Open joint type revetments are suitable in situations where river water level and or phreatic surface within the bank fluctuates significantly either seasonally or diurnally because in combination with suitable filter media, releasing of hydrostatic pressure is effectively possible.

2. Close joint type in witch joint gaps between individual hard materials are sealed using cement, bitumen, or asphaltic materials. Close joint type revetments do not allow free flow of water and are liable to develop detrimental head of water behind witch materials eventually lead to blow-offs.

Close-joint type revetments are suitable only in situations where fluctuations in water level (river or phreatic) are either insignificant or not of concern e.g. upper bank above HWL.

Structural Components of Revetment

A revetment normally consists of a cover layer, which provides protection against erosion forces generated by the flow, wave action and mechanical impacts. Below the cover, a filter layer is required above the underlying soil to protect the structure from seepage effects, surface runoff and down slope migration of soil particles. Toe protection is provided at the foot of the bank to prevent undercutting caused by scour. The falling apron and launching apron are two parts of it. Figure 2.1 shows revetments and their different components.

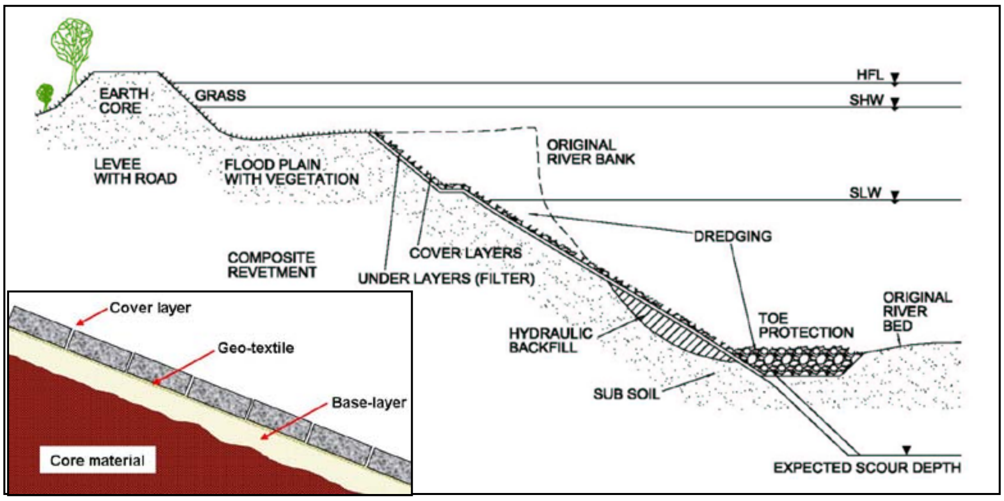


Figure 2.1 Different components of a revetment work

Different Types of Revetment Protection Works

Different types of materials are used for slope protection. In general, revetments can be grouped as:

1. Self-adjusting structures- including all kinds of rip-rap protection using stones, concrete blocks etc. but also hand-laid bricks and concrete block layers without interlocking, slurry-filed geotextiles bags etc.

2. Flexible structures- including articulated blocks and slabs, wire mesh netting and other forms of mattresses, tubes, gabions and interlocking block layers.
3. Rigid structures- including asphalt and concrete paving, concrete filled pillows, grouted mattress, grouted rip-raps, soil stabilization etc.

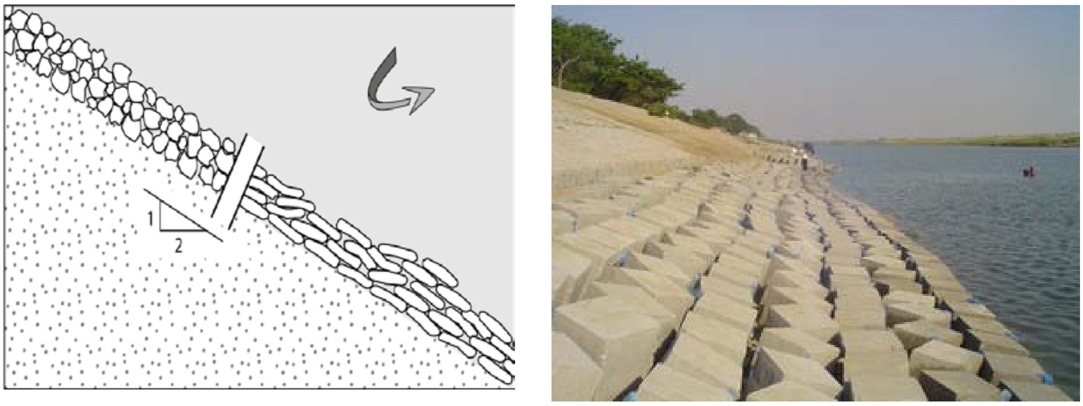


Figure 2.2 : Different types of self adjusting revetment: multi-layer riprap and geobags (left), loose cement concret blocks (right)

Based on construction materials, revetment may be of different types: rock riprap, rubble riprap, wire riprap, wire enclosed rocks, pre-formed blocks, grouted rocks or paved lining etc. Material units used for cover or armour layer are (a) stones or boulders, (b) concrete blocks or slabs, (c) various bituminous system and (d) fabric containers. Different practices of using these materials are explained below.

Protection Measures with Rip-rap

Rip-rap is one of the most common types of armour layer which comprises randomly placed quarried rocks. It is made up of durable stone, sizes ranging typically from 10 to 50 cm. Usually it is specified by mass and typically ranges from 10 to 500 kg. Rip-rap is usually placed in one, two or three layers. Their shape depends on rock quality and can be rounded, cubical, tabular, angular or elongated. Angular, near cubical shaped stones are most suitable for riprap construction. In Bangladesh, boulders are normally collected from the north-eastern rivers. Quarried stones are not locally available at large quantity. Potential sources of quarried rocks are Chittagong Hills and Madhayapara hard rock mine. Currently, assorted sizes of quarried rocks are available from the later source

Protection Measures with Pitched Stone

The pitched stones or hand-pitched stones are entirely placed by hand as a single layer. Normally, approximate single-sized stones are used only. The shape of stone should be nearly cubical and the ratio of length to breadth should not exceed two. Pitched stone can only be installed above water level, therefore, as part of other revetment system only. Pitching is normally placed on an appropriate filter layer. In case of high current or wave attack, pitched stones are placed on cement mortar.

Protection Measures with Grouted Stone

In order to increase the stability of rip-rap or pitched stones a cement grout or bitumen can be applied. This system is more stable to wave and current attack than loose stones. The permeability is reduced in this case and the stability may be adversely affected by excess pore pressure during ground water flow towards the bank.

Protection Measures with Loose Blocks

Pre-cast concrete blocks generally without reinforcement are designed for a specified minimum strength with durable aggregates. Different shapes like cubical, cuboidal etc. are used. Loose blocks can be hand-placed above the low water level and dumped below low water level like rip-rap (Figure 2.3).An approximate sub-layer or filter layer is required. The concrete blocks are widely used in Bangladesh due to the fact that the blocks can be made on-site using local non-technical persons with reasonable cost.

Figure 2.3 : Concrete blocks used in revetment

Protection Measures with Concrete Slabs

Use of plain concrete or reinforced concrete slabs, pre-cast or cast in situ, is also another option for cover layer. Their applicability is, however, limited since they are normally built or placed in the dry condition. The use of pre-cast concrete slabs above low water level is more convenient if placed on an appropriate filter. Concrete slabs have been used on embankment slope in Bangladesh against wave erosion.

Protection Measures with Interlocking or Articulated Blocks and Slabs

The stability of a cover layer consisting of concrete blocks can be improved by interlocking geometric shapes. Additional stability is attained by mobilizing the weight of adjacent blocks which increases the stiffness of the cover layer. Placing by hand or by mechanical means is possible depending on the system, even under water. Although many articulated concrete block systems are in use in different countries, these are not widely used in Bangladesh.

Protection Measures with Cable Connected Blocks

Placing of concrete blocks under water may become more systematic and quicker in case the individual blocks are held together by cables of steel or synthetic material reducing the risk of localized failure. All types of cable-connected blocks are to be anchored by appropriate means at locations beyond potential failure planes which may be difficult to achieve in practice.

Bituminous Systems Protection

Bituminous systems provide a flexible cover layer which can withstand substantial hydraulic loadings. Special attention must be given to the durability of the bituminous revetments because bitumen hardens with exposure to ultra-violet radiation and the atmosphere. Furthermore, abrasion can be caused by floating material, which can even create damages to the revetments above the waterline. Asphalt mixes may be designed to be either permeable or impermeable. Bituminous system is not used for bank protection in Bangladesh except that use in the Jamuna Multipurpose Bridge Project.

Protection Measures with Bags

Jute bags, synthetic bags or geo-textile bags (woven or non-woven needle punched) filled with ballast can be placed in one to three layers directly on a slope or along the toe of the eroding slope or on eroding river bed. The type of ballast is to be selected considering the required flexibility and permeability of the cover layer. Sand and gravel filled bags are relatively flexible.

The non-woven needle punched geo-bags filled with sand is one of the very useful material for protection of riverbank. Fine sand available in the river bed filled in geo-textile bags of desired size, can be effectively used for protection of vulnerable stretches of the riverbank. In this approach, the slope below low water level is protected with geobags. The slope above it is protected with cement concrete blocks mainly to resist wave action. The bags are filled with sand and weight 78 kg, 126 kg and 250 kg. Currently, geobags are used both for emergency and permanent protections.

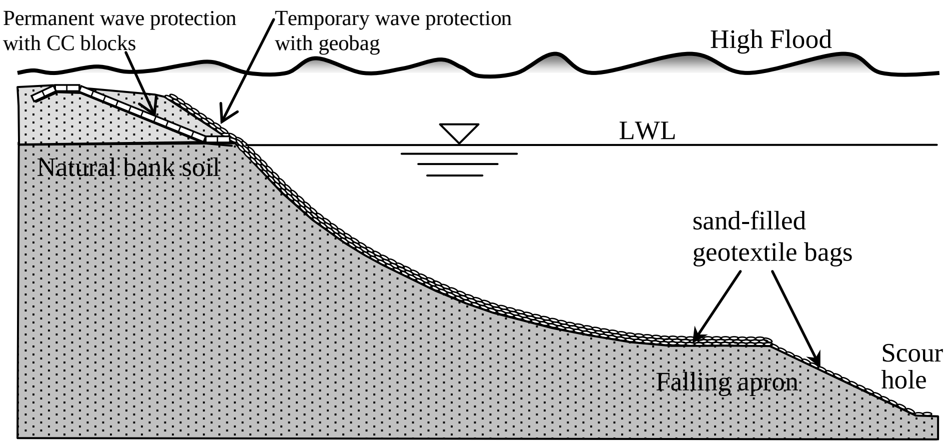


Figure 2.4 **:**Typical Geobag revetment (cross-section)

Groynes are structures built at an angle to the river bank to deflect flowing water away from downstream critical zones to prevent erosion of bank. Groynes may be constructed for multiple purposes to control erosion, to maintain safe navigation channels as well as to reduce the flow velocity.

A typical groyne has three parts: head, shank and root. The riverside end of the groyne is the head of the groyne, the joint of the groyne with the existing bankline is the root of the groyne and the longer part joining the head and root of the groyne is the shank. Groynes can be classified based on

(a) Orientation to the flow
(b) Shape of the groyne head and
(c) Permeability of the structure

A groyne placed at right angle to the bank (perpendicular flow attack) is termed as deflecting groyne. Groynes inclined in upstream direction are called repelling groynes and groynes pointed downstream to attract the flow towards the structure's head and thus to the river bank is called attracting groynes

Different types of groyne heads of impermeable and unsubmerged groynes are described below and shown in sketches of Figure. 2.5.

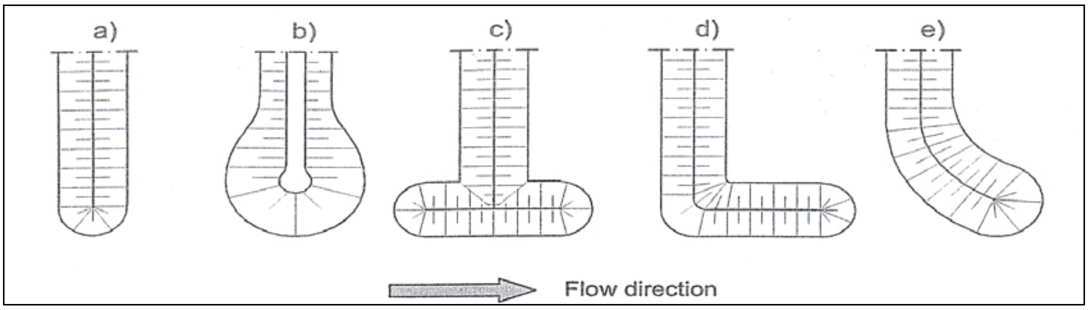


Figure 2.5 Different types of groyne heads: (a) straight, (b) bell-head, (c) T-head, (d) L-head, (e) Hockey-head

Permeable groynes are generally constructed of timber, steel or reinforced concrete piles driven or sunk into the river bed in one or several rows. Impermeable groynes can be built of local soil, stones, gravel and rock with suitable slopes at the shanks and the head or even vertical walls at the shanks, using sheet piles. In case of an appropriately sloped earth dam, the shank and the head have to be protected by a cover layer placed on a suitable filter layer

These are structures extending from the bank into the river to intervene with flowing water in ways conductive to predetermined purposes in relation to the river training. Spurs/Groynes can be constructed of timber, steel or concrete piles and of earth, rock boulders, solid constructions are sometimes called Groynes.

Spurs/Groynes can be different types. Classification of spurs/groynes is also done in different ways depending on different aspects or features of the structures as indicated below.

a) Based on submergence criteria, spurs/groynes are classified as:

- Submersible spurs which are of low height and often go under water. Top profiles of submersible spurs may be either horizontal or sloping towards the river.

- Non-submersible spurs/groynes which are of adequate height such that they will not be submersed even at the highest flood condition in the river.

Submersible spurs have the inherent disadvantage that they induce scour at the downstream due to flow of water over them. They are sometimes dangerous to river traffic. However, they are suitable for rivers carrying floating debris.

2.7.4 Causes of failure of bank protection measures

a) Failure by surface erosion of riverbanks

Surface erosion of river banks occurs if the driving erosive forces are exceeding the resistive forces of the individual grains or of the conglomerates in case of cohesive materials. The main impacts responsible for surface erosion at river banks are :

a) Current induced shear stress
b) Wave loads (wind-generated waves; ship and boat-generated waves)
c) Seepage (excessive pore pressure)

d) Surface runoff and

e) Mechanical action (desiccation, ship impact, activities of humans and animals)

b) Failure of protective works by mass damage of riverbanks

Mass damage of river banks can be divided into slip failures, block failures and flow slides, initiated by different processes, which are illustrated in Figure 2.6. The actual damage of a river bank may not follow immediately after an impact. In some cases the failure process takes several days. On the other hand, a damage may occur without warning at almost any time if active surface erosion and toe scouring is prevalent or an additional (surcharge) load is applied to the bank. The risk of mass damage is increased during heavy rain and during quick fall of river stages after flood. Potential failure modes of slip and block failures are listed below:

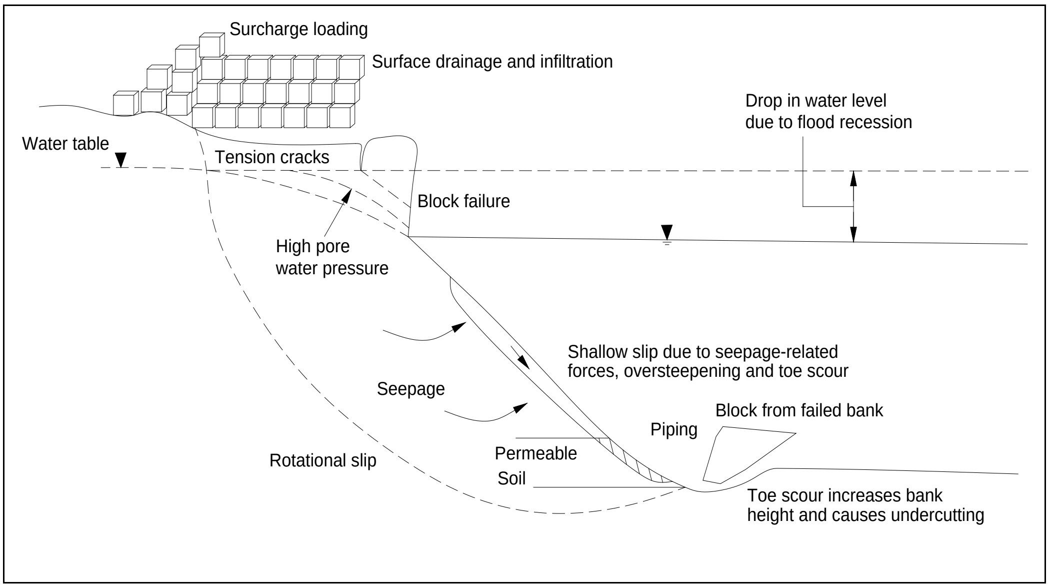


Figure 2.6 : Processes responsible for mass failure of a river bank

Slip failures

- In case of non-cohesive material and a shallow bank angle the failure surface is usually approximately parallel to the slope angle. Water seepage can substantially reduce the stability of the bank, whereas vegetation will normally help to stabilize against failure.

- Steep or almost vertical banks of non-cohesive material can fail along a plane or slightly curved surface. This is often the case when the river water level is low relative to the total bank height.

- If relatively deep tension cracks have developed on the surface of the river bank, failure occurs by sliding and/or toppling. This failure mode is little affected by the groundwater table, but is more likely if the crack fills with water.

- Deep-seated rotational failure is possible in cohesive soil where the banks are steep and moderately high. If the soil is relatively homogeneous, the failure surface may follow a circular arc.

- It is also possible that layers of weak material affect the actual shape of the failure surface, which may then include logarithmic spirals or even planar sections. Both types of failure can extend beyond the toe of the bank. The stability is significantly affected by the position of the water table and if the tension cracks are filled with water.

- If the outside of an eroding meander bend lies at the edge of the river valley, further erosion can trigger a massive landslide stretching up the valley slope. Tension cracks, bulging above the toe or noticeable movement are signs of potential failure.

Block failure

- If the lower part of a composite bank, which is more frequently exposed to flowing water, consists of more erodible material such as sand and/or gravel, the upper part can be under-cut and falls as a complete block down the slope.

Typical modes of river bank failure are exemplary shown for slip failures and for block failures and for block failures in Figure 2.7(adapted from Hermphill and Bramley, 1989.).

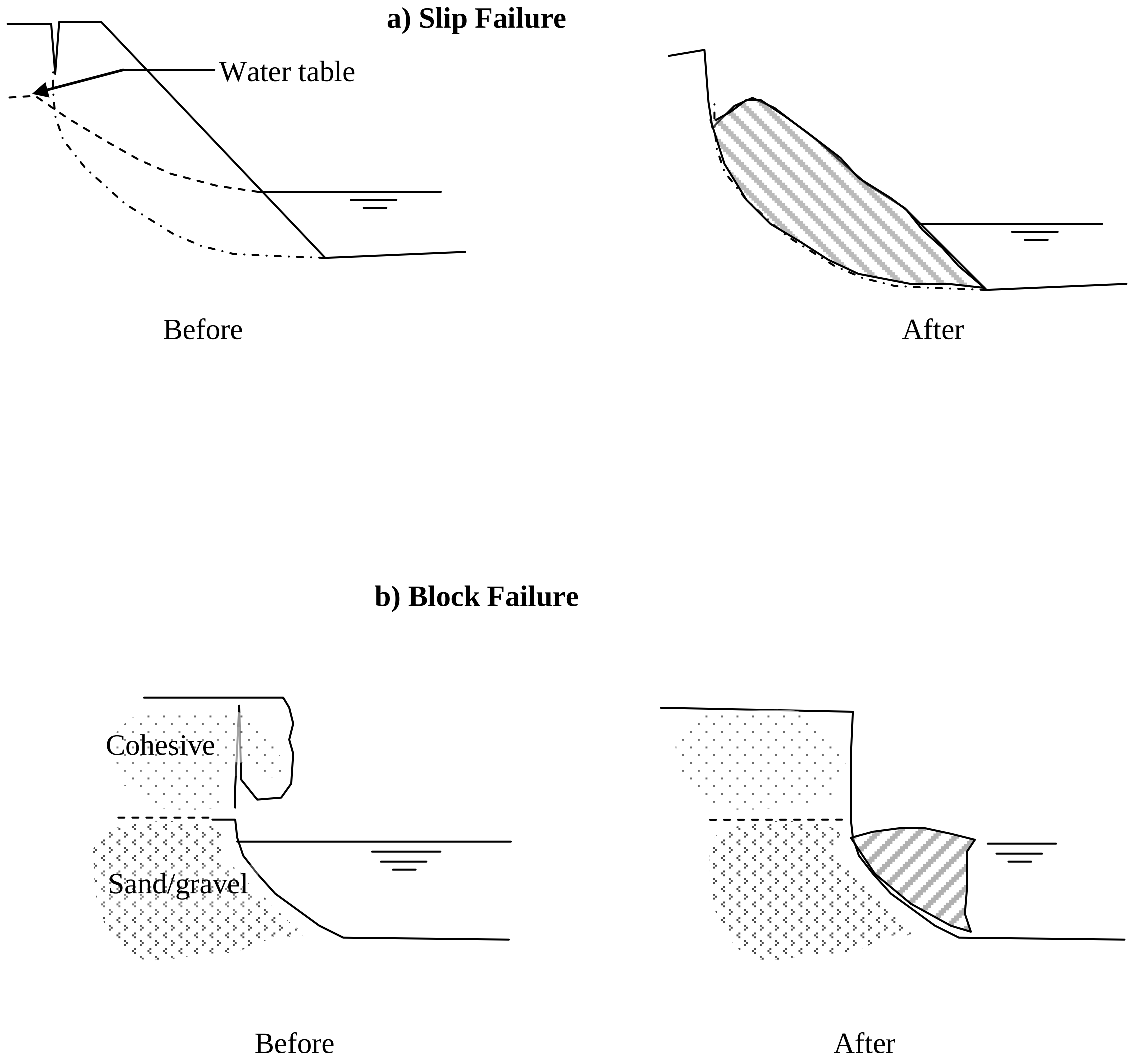


Figure 2.7 :Typical Riverbank Failure

Apart from the causes described above, the following causes may also be worth mentioning here in connection with the cause of bank failure.

1) Failure of the toe which is usually caused by :

a) insufficient depth of protection

b) insufficient weight of the protecting materials

c) non-flesibility of the mattress.

This kind of failure is generally experienced at the heads of groynes or bank heads, or along a bank revetment work including retaining walls.

2. Bulk head and groynes may fail by outflanking due to :

a) insufficient protection at the wing

b) embayment by the river further upstream.

Long groynes may be breached at their middle portion owing to infringement by a sharp turning current.

3. Paved slopes themselves may fail because of one or more of the following causes

a) inadequate drainage at the back of the bank

b) insufficient depth of paving

c) inadequate depth of filter under paving

d) insufficient bond between upper bank paving and protection

e) dislodging of mattress or paving blocks

f) piping out of soils from underlying soil due to inadequate filter blanket.

CHAPTER THREE

3. APPROACH AND METHODOLOGY

The approach and methodology that has been used in this study are described below.

3.1 Field Visit and Data Collection

A number of field visits have been made in order to know the existing condition of the protective works at Titporol and Debdanga as well to collect the recent data particularly needed for the technical evaluation of the implemented bank protective works at Titporol. During field visit the following data has been collected:-

i) Index map of the study area
ii) Cross-section of the river along the bank protective work at Titporol before and after slope failure
iii) Hydraulic and hydrological design data
iv) Bed sediment and Bank soil information around the study area.

3.2 Analysis of Data and Review of Design

The data that are collected during the field visits are analyzed and used for the review of design procedure of the protective works employed by BWDB. The review of design includes the evaluation of cover layer design, filter design and toe protection design under hydraulic load and anticipated scour condition. Pilarczyk formula has been used for assessment of cover layer of protective works and available scour formula has been used for scour calculation. Observed scour values has been compared with the predicted scour. In addition, attempts were also made to identify the possible causes of the failure of the slope and other damages by checking the stability of slope. As the stability of slope depends very much on the properties of the underlying soil of the slope, the bore-log data collected from the field office has been analyzed. Slope stability analysis has been carried out using a computer program widely used in considering different options such as with different river stages and seepage forces, different slopes of protective works etc.

3.3 Interaction with BWDB field Officials

An interaction meeting was made during the course of this study with BWDB field officials in order to get necessary information regarding the constructional matters such as time and schedule of construction and their views of the failure of the protective work at Titporol.

3.4 Comparison With other Effective Bank Protection Site at Debdanga

Similar analyses mentioned above were made for the protective works at Debdanga considering it as an implemented effective protective works. Comparison of the results of two cases is made to identify the cause of failure at Titporol.

3.5 Design of Protection Works

In this chapter, the hydraulic and hydrologic design parameters for bank protection has been discussed. These design parameters are the design discharge, the maximum, minimum and standard low water levels, flow velocities, wind speed and wind waves, design scour depths and morphologic changes. Design procedure for the protective works and different approaches for the geobag as a river bank and bed protection material has also been described here.

3.5.1 Consideration for water levels

The Design Water Level (DWL) is related to PWD and can be derived from the design discharge. For the design of bank protection structures maximum, minimum, bankfull and average low water levels are very important. The design water levels shall be determined at the proposed protection work site, from the water levels available at some reference stations and the water surface slope applicable for that sequence. The design water levels may be taken from available analyzed records or may be computed from available data.

Low water level (LWL) is particularly important for fixing under water river training works. Mainly the average high water occurring in the lean period (December-March) shall be the focus for fixing the DWL.

The design water levels (DWL) are the calculated water levels based on measurements for the last four decades. It is assumed that the observed water levels are not influenced by wind set-up and transverse gradients in a cross-section. The design Water Level (DWL) is related to PWD dat6um and can be derived from the design discharge. As the relation between discharge and water level varies due to rapid morphological changes, only a stage discharge relation (rating curve) established at the location of the planned structure from long term monitoring of daily averaged water levels and corresponding discharges can be used.

River levels for both flood and dry season play important role in flood control and river training works. For the design of bank protection structures maximum, minimum, bank full and average low water levels are very important. The design water levels shall be determined at the proposed protection work site from the water levels available at some reference stations and the water surface slope applicable for that sequence. The design water levels may be taken from available analyzed records or may be computed from available data. The following end results of hydrological analysis are important for the design :

Flood Levels: 1:100 year
1:50 year
Bankfull level

Low Water Level: Lowest during dry season (December-March)

Low water level (LWL) is particularly important for fixing under water river training works. Design Low Water Level (DLWL) in consideration of construction and maintenance of bank protection works shall ventilate the safe construction window available in average years. Mainly the average high water occurring in the lean period (December-March) shall be the focus for fixing the DLWL.

Following points may be considered in fixing the LWL:

- High value of adopted LWL will decrease the length of slope protection but will increase the value of D_s that will increase the length of falling apron,

- Adaptation of too low value of LWL may create a situation that the level may not be achieved in the year of construction.
- The idea for time available for construction (construction window) should there ore be such that some flexible planning may be done well before the construction period.

Selection of DLWL:

(I) Non Tidal Area

- In order to make a balance between too high and too low value of LWL it is proposed that Upper Quartile value of annual LWL or a value in consideration of construction window be adapted as the DLWL for the design.

(II) Tidal Area

- Instead of annual lowest WL, annual lowest high tide (ALHT) should be used and hen the Upper Quartile value of this may be used. Forecast of a fortnightly tide profile covering both neap tide and spring tide should also be provided so that the information may be used as an aid to under water construction planning

3.5.2 Consideration for velocity

The design flow velocity may be determined by average flow velocity and from physical model investigation. Average flow velocity in a cross-section of a channel is estimated with a regime equation. Design flow velocities from measurement in a physical model investigation depend on the approach flow and on the alignment of bank protection structure. In general the depth averaged flow velocity is used as design flow velocity.

The design flow velocity may be determined according to the following approach:

- Design flow velocities are obtained either from field measurement or in a Physical model investigation. These design flow velocities depend on the approach flow and on the alignment of bank protection structure

- Average flow velocity in a cross-section of a channel is estimated with a suitable equation.

The flow velocity obtained from the method stated above shall be verified with the observed measurements available for that stretch of the river. For the design of drag force on the piles and bed protection around the piles of permeable groynes a designed flow velocity is defined as the upstream flow velocity which is not influenced by the permeable groynes. In general the depth averaged flow velocity is used. The flow attack on a revetment depends not only on the discharge and the water level but also on the alignment of approach channel. The design flow velocities are the maximum flow velocities measured on the physical model with the extreme alignments of the approach channel.

Flow fields in the main rivers, however, are determined by the bed topography, which is determined by the morphological processes. Average velocities in the Jamuna can be evaluated using at-a-station relationships (Klaassen & Vermeer, 1988), resulting in:

$$u=0.095\ Q^{o.26} \qquad (3.1)$$

A similar approach for the Ganges on the basis of the at-a-station relations derived in RSP special report 7 (1996), results in:

$$u=0.36\ Q^{o.15} \qquad (3.2)$$

where u is the average velocity and Q is the bankfull discharge.

For a bankfull discharge of 44,000 m3/sec, this results in an average velocity of about 1.5 m/sec for Jamuna and 1.8 m/sec for the Ganges. Locally the velocities may be much higher due to dimensional effects. Maximum velocities measured were (FAP-24):

- 3.2 m/sec in an eroding outer bend in Jamuna near Kamarjani,
- 3.7 m/sec near a protrusion in the Jamuna near Bahadurabad
- 4.0 m/sec near a protrusion in the Ganges, upstream of Gorai offtake.

3.5.3 Consideration for discharge

The design discharge can be taken from available analyzed records or may be computed from available data. The discharge of a specific river is obtained from the analysis of hydrological data, especially through extrapolation of stage-discharge relations at water level stations, where the discharge measurements have also been executed regularly. The analysis of flood discharges and the associated recurrence periods result in a probability function which can be used to define the design discharge with a return period of 100 years. The discharge of the design flood is defined in order to estimate the Design Water Level (DWL) with a return period of 100 years. Additionally, a bankfull discharge is defined for the estimation of the design cross-section.

According to Inglis,

$$Q_{bankfiill} = Q_{dominent} = 2/3*Q_{max}$$

For example if

$$\text{Maximum Discharge, } Q_{max} = 1{,}010 \text{ m}^3/\text{s}$$

$$Q_{bankfull} = Q_{dominent} = 2/3*Q_{max} = 673.33 \text{ m}^3/\text{s}.$$

3.5.4 Consideration for waves

Waves are defined by wave height H, wave period T, wavelengths L, and direction. Waves are often generated far from the place where they are observed. However, related wind speed and duration can be derived from the observed wave height, wave period and wave direction.

Waves at the river training sites should either be generated by wind or by water vessels. wind waves would usually govern the design of protection work at the slopes of river training works. With respect to protection, two aspects of waves have to be considered:

- Run-up of waves against slope which might overtop the upper limit of protection,
- Erosive forces of breaking waves against the slope causing erosion.

The principal factor affecting the design of slope protection is wave action. The mechanics of wave generation are extremely complex, and the forces causing erosion during wave attack on an earth slope are both varied and complex. The described ranges of riprap design assume that the wave height is a direct measure of the erosiveness of the wave.

To evaluate wave height the following factors that create waves are to be analyzed:

- Design wind direction,
- Effective fetch,
- Wind speed and duration.

The mechanism of formation of wind generated wave and its relation/dependence with duration of wind, wind speed, fetch length, depth of water and other phenomena have been described in following sections.

Generation of Waves

The generation of waves depends on fetch length, wind speed and duration of wind. The fetch length is the length of the water surface, for example of a lake the wind is blowing across. It is the length of the water surface, where the wind can transfer energy to the water. Wave generation can be limited by (i) the duration of the occurring wind (duration limited) or (ii) the length of the water surface of lake (fetch limited). When a wind blows, with essentially constant direction, over a fetch for sufficient time to achieve steady-state, it is termed as fetch-limited values. The second idealized situation occurs when a wind increases very quickly through time in an area removed from any close boundaries. In this situation, the wave growth can be termed duration-limited. It should be recognized that this condition is rarely met in nature; consequently, this perdiction technique should only be used with great caution.

Wind must blow for a certain time to develop the full wave height for the given fetch length. Only after some time of blowing across the surface, sufficient energy is transferred into the water surface to generate the full wave height. In rivers, lakes and estuaries fetch determines the wave condition and not the duration of the wind (Alam and Fontijn (2006). So design of bank protection works in rivers will be governed by fetch limited wave conditions.

Data for calculating wave heights

Wind direction: wind direction can be obtained by determining the point on the shoreline over the longest stretch of open water from the embankment. The direction should be weighted with other topographic conditions or climatic information.

It is hardly possible to give firm prediction of the directions in which wind generated waves would be progressing. For design purposes it will have to be presumed that they will approach any river training work perpendicularly.

Effective Fetch (F_e)

Early studies on wind and wave development assumed the fetch to be the greatest straight-line distance over the open water. Subsequent studies by Seville (1952) showed that the shape of an open water area affects the fetch, the smaller the width to length ratio, the smaller the effective fetch. Effective fetch can be determined from climatological data or from site conditions.

Wind Speed and Duration (U_d & T_d)

Reliable estimate of the maximum wind speed that would exist over a length of time at a given site is practically impossible. However, value of wind speed can be obtained from climatological data of the area for cyclones, thunderstorms or norwester and design wind speed may be selected for appropriate return period.

Significant Wave Height (Hs)

The significant wave height is the average height of the highest one-third of the waves for a specified period of time.

Waves in Rivers

The flow velocity of rivers has to be taken into account for calculating the wave height. The wind is transferring energy into the water based on the sheer stress. If the wind speed and the flow velocity have the same direction the sheer stress becomes less, because the difference in velocity is less. To calculate the wave heights for these cases the flow velocity has to be deducted from the wind speed if they both have the same direction. If the direction is opposite to each other the velocities need to be summed up.

For the major rivers in Bangladesh the highest measured flow velocities over longer areas are 2 m/s close to the surface even though locally peaks of up to 4m/sec can be observed. Assuming a fetch length of 5 km and a wind speed of 17 m/sec acting for 20 minutes on the river surface, the generated wave will have a height of 0.7 m and a wave period of 2.1 sec. Superimposing the flow velocity u = 2 m/s and the wind speed of v = 17 m/s, calculated wave height is 0.8 m with a period of 2.2 sec. If the wind comes from the same direction then the wave height is 0.6 m with a period of 2.0 sec.

3.5.5 Stability of revetment under current attack

Different methods regarding calculations of unit dimensions of revetment cover layers and toe protections show only marginal deviations within the range of application for the rivers of Bangladesh. Therefore, the widely used Pilarczyk method (1990) is used because it includes the turbulence intensity by an empirical coefficient.

(1) Pilarczyk Method

The general formula for the design against current loads is:

$$D_n \geq \frac{0.035 \cdot \overline{u}^2}{\Delta_m \cdot 2g} \cdot \frac{\phi_{sc} K_\tau K_h}{K_s \cdot \Psi_{cr}} \qquad (1)$$

Where,

D_n (m) nominal thickness of protection unit (cover layer)

• $_m$ (-) (• $_s$-• $_w$)/• $_w$ = relative density of submerged material

• $_s$ (kg/m^3) density of protection material

• $_w$ (kg/m^3) density of water

• (m/s) depth averaged flow velocity; if replaced by u_b= 0.6• , a value of K_h = 1, must be applied,

g (m/s^2) acceleration due to gravity

ϕ_{sc} (-) stability factor

K• (-) turbulence factor

K_h (-) Depth factor, dependent on the assumed velocity profile and water depth (h) to equivalent roughness height ratio: $K_h = (h/Dn+1)^{-0.2}$; for a non developed velocity profile.

K_s (-) $K_s = \sqrt{1-\left(\frac{\sin\alpha}{\sin\theta}\right)^2}$

• $_{cr}$ (-) critical Shield's parameter,

• (°) slope angle of bank or structure

• (°) angle of repose considering the material specific internal friction

The stability parameter ϕ_{sc} depends on type of application, some guide values are given in Table 3.1.The degree of turbulence can be taken into account with the turbulence factor K• and some guide values for K• are given in Table 3.2.

Table 3.1 Values of stability factor

Revetment Type	**Stability factor ϕ_{sc}**	
Cover layer	**Continuous protection [-]**	**Exposed edges transitions [-]**
Riprap and placed blocks	1.0	1.50
Block mats, gabions, washed-in blocks, geobags and geomattress	0.5	0.75

Source: Pilarczyk, K.W. (1998)

Table 3.2 Turbulence Intensity Factor K. (current)

Turbulence Intensity	K. (-) Gabions, Mattresses	K. (-) Others
Normal turbulence in rivers	1.0	1.0
Non-uniform flow with increased turbulence, mild outer bends	1.0	1.5
High turbulence, local disturbances, sharp outer bends	1.0	2.0

Source: FAP-21, (2001)

With the depth parameter $K_{h,}$ the water depth is taken into account, which is necessary to translate the depth averaged flow velocity into the flow velocity just above the revetment. According to Pilarczyk (1998) for a non developed velocity profile K_h factor can be expressed as

$$K_h = (h/D_n+1)^{-0.2} \text{ and} \qquad (2)$$

$$K_h = 1.0. \text{ (For very rough flow, } h/k_s < 5\text{)}$$

A set of values for angle of repose • for different materials used as protection element on different types of filter material and their density have been proposed in the Guidelines and Design Manual for Standardized bank Protection Structures, FAP 21 is shown in Table 3.3.

Table 3.3 Angle of repose • and density $•_s$ for various revetment cover layers

Revetment type		Angle of repose • [°]	Density of protection material, $•_s$ [kg/m^3]
Cover layer	**Filter material**		
Randomly placed, broken rip-rap and boulders	Geo-textile Granular	25 30	2650
CC blocks (cubical shape), randomly placed in multi layer	Geo-textile Granular	30 35	2250
CC blocks (Cubical shape), hand placed in single layer	Geo-textile Granular	20 25	2250
Gabions/mattress filling by stones		45	2650

Source: FAP-21, (2001).

Some guide values for critical Shield's parameter $•_{cr}$ are given in Table 3.4.

Table 3.4 Critical Shield's Parameter • $_{cr}$

Revetment Type	Critical Shield's Parameter • (-)
Riprap, small bags	0.035
Placed blocks, geobags	0.05
Block mats	0.07
Gabions	0.07 (to 0.10)
Geomattress	0.07

Source: FAP-21, (2001).

The other formulae used for this purpose are :

(2) Isbash Formula

$$W = \frac{4.1\times 10^{-5} \cdot S_s . V_5}{(S_s - 1)^3 \cdot \cos^3 \alpha} \tag{3}$$

(3) California State Highways

$$W = \frac{2\times 10^{-5} \cdot S_s \cdot V^6}{(S_s - 1)^3 \cdot \sin^3 (70 - \alpha)} \tag{4}$$

(4) PIANC

$$D_n = \frac{0.70\, V^2}{g\,(S_s - 1) \cdot \cos\alpha \left(1 - \tan^2\alpha / \tan^2\theta\right)} . \tag{5}$$

(5) JMBA

$$D_n = \frac{0.7V^2}{2 \cdot (S_s - 1) \cdot g} \cdot \frac{2}{\log(6h/D)^2} \cdot \frac{1}{\left[1 - (\sin\alpha / \sin\theta)^2\right]^{0.5}} \tag{6}$$

(fps or metric unit)

(the equation has been developed for cc blocks)

In the above equations:

W	(kg or lb)	weight of individual stone
D	(ft or m)	diameter of stone
D_n	(ft or m)	dimension of cube
V	(ft/s or m/s)	mean velocity at the adjacent channel
h	(ft or m)	depth of water
S_s	(-)	specific gravity of stone
•	(°)	slope of bank
•	(°)	angle of repose of revetment material
g	(m/s^2 or ft/s^2)	gravitational acceleration

3.5.6 Stability of Revetment under Wave Attack

The wave attack is considered significant for the revetment cover layer above low water level and some parts of the falling apron. The minimum dimensions for the stability of the cover material under wave attack can be determined by Pilarczyk formula and Hudson's formula. The Pilarczyk formula being more universal with its breaker similarity index is preferred.

(1) Pilarczyk Formula

$$D_n \geq \frac{H_s \cdot \xi_z^b}{\Delta_m \cdot \Psi_u \cdot \phi_{sw} \cdot \cos\alpha} \qquad (7)$$

where,

Dn [m] characteristic size of the revetment cover layer (single unit

		size for loose elements, thickness for mattress systems)
Hs	[m]	significant wave height
$\bullet_{m}$	[-]	relative density of submerged material = $(\bullet_{s}-\bullet_{w})/\bullet_{w}$
g	(m/s^2)	acceleration due to gravity (= 9.81)
$Ø_{sw}$	[-]	stability factor for wave loads
$\bullet_{u}$	[-]	system specific stability upgrading factor
•	[°]	bank normal slope angle
•z	[-]	wave breaker similarity parameter= $\tan\alpha \cdot \frac{1.25 \cdot T_m}{\sqrt{H_s}}$
T_m	[s]	mean wave period
b	[-]	wave structure interaction coefficient, dependent on roughness and porosity of protective material

The formula is valid for •z <3 and cot•• 2. The material and armour layer unit specific coefficients to be applied for design against wave attack are summarized in Table 3.5.

Table 3.5 Coefficients for design of various cover materials against wave attack

Revetment type	**Stability factor for incipient motion ϕ_{sc} [-]**	**Stability upgrading factor, $\bullet_{u}$ [-]**	**Interaction coefficient, b [-]**
Randomly placed, broken riprap and boulders	2.25-3.00	1.00-1.33	0.50
CC blocks, cubical shape, randomly placed in multi-layer	2.25-3.00	1.33-1.50	0.50
CC blocks, cubical shape, hand placed ,single layer (geotextile filter)	2.25	2.00	0.67 • 1.00
CC blocks, cubical shape, hand placed in single layer, chess pattern (geotextile on sand)	2.25	1.50	0.67 • 1.00
CC blocks cable connected	2.25	1.80	0.67
Wire mesh mattress	2.25	2.50	0.50
Gabions/mattress filling by stone	2.25	2.50	0.50

Source: BRTC, BUET, (2008)

3.5.7 Thickness and Grading of Riprap

Riprap Thickness

Opinions of different authorities regarding the thickness of slope pitching are given below:

1. U.S. Army Corps of Engineers (1991), recommends that thickness of protection should not be less than the spherical diameter of the upper limit W_{100} (percent finer by weight) stone or less than 1.5 times the spherical diameter of the upper limit W_{50} stone, whichever results in greater thickness.

2. California Highway Division (1991) recommended that there should be at least two layers of overlapping stones so that slight loss of materials does not cause massive failure.

3. ESCAP (1973) recommends that thickness of protection should be at least 1.5D, where D is the diameter of the normal size rock specified.

4. Spring (1903) recommended thickness of stone in inches for covering rough, heavy and loose stone for pitching from low water upwards as shown in Table 3.6.

5. The thickness of stone pitching and soling for permanent slopes required at head, body and tail of guide bank for river flowing in alluvial plains as recommended by Gales (1938) is given in Table 3.7.

6. Inglish (1949) recommended following formula to compute thickness of protection required,

$$t = 0.06\ Q^{1/3} \tag{9}$$

Where, t = thickness of stone riprap (m)

Q = discharge (m^3/s)

Inglish's formula apparently gives excessive thickness for higher discharge. The thickness suggested above should be increased by 25% when the riprap is placed under water to provide for uncertainties associated with the type of placement

Table 3.6 Spring's Table to Compute Thickness of Stones on Slope

River bed materials as classified by Springs	Thickness in inches for river slopes in inches per mile					Remarks
	3	**9**	**12**	**18**	**24**	
Very Coarse	16	19	22	25	28	The stone pitch prevents sand underneath from being sucked out by high velocity. More rationally stone pitch thickness should be based on velocities.
Coarse	22	25	28	31	34	
Medium	28	31	34	37	40	
Fine	34	37	40	43	46	
Very Fine	40	43	46	49	52	

Source: BRTC, BUET, (2008)

Table 3.7 Gale's Table to Compute Thickness of Stone on Slope

River	Rivers with discharge 0.25 to 0.75 million cusec		Rivers with discharge 0.75 to 1.50 million cusec		Rivers with discharge 1.50 to 2.50 million cusec	
Parts of guide bundh	Head	Body and Tail	Head	Body and Tail	Head	Body and Tail
Pitching stone	3•-6•	3•-6•	3•-6•	3•-6•	3•-6•	3•-6•
Thickness of soling ballast	7•	7•	8•	8•	9•	9•
Total thickness	4•-1•	4•-1•	4•-2•	4•-2•	4•-3•	4•-3•

Source: BRTC, BUET, (2008)

3.5.8 Considerations for Filter Material

Geotextile is use as a filter material. The main design parameters for geotextile filters are the retention criterion and the permeability criterion, which define the capability of the material to retain the subsoil without clogging and to allow undisturbed water transport through the membrane.

3.5.9 Determination of the Range of Grain-size Distribution

The grain-size distribution curve must be determined following international standard regulations, to allow for calculation of the various design parameters. As the filter characteristics of geotextile are mainly influenced by the fine compartment of the grain-size distribution, the PIANC method categorizes the soil by the screen fraction smaller than 0.06 mm grain size. Soil categories for geotextile filter design are classified as follows:

Range A: 40% or more of the soil particles are smaller or equal to 0.06 mm.
Range B: 15% or less of the soil particles are smaller or equal to 0.06 mm.
Range C: between 15% and 40% of the soil particles are smaller or equal to 0.06 mm

3.5.10 Design of Toe Protection

At the estimated maximum scour depth, the launching apron is assumed to cover and stabilize the bank-sided river profile, preventing from further erosion of the bank. The method had been widely used on sand bed streams. Scour depth R can be calculated by Lacey's formula which is given below

$$\text{Scour Depth } R = 0.47 * (Q_d / f)^{1/3} \quad (10)$$

Where,

Q_d = Design Discharge

f = Silt factor = 1.76 x • D_{50}

a) Thickness and shape of Launching Apron

According to Spring (1903), minimum thickness of apron is equal to 1.25 times the thickness of stone riprap of the slope revetment.

According to Rao (1946, after Varma, Saxena and Rao 1989), thickness of apron at junction should be 1.5 times the thickness of riprap in slope. Thickness at river end of apron in such case shall be 2.25 times the thickness of riprap in slope.

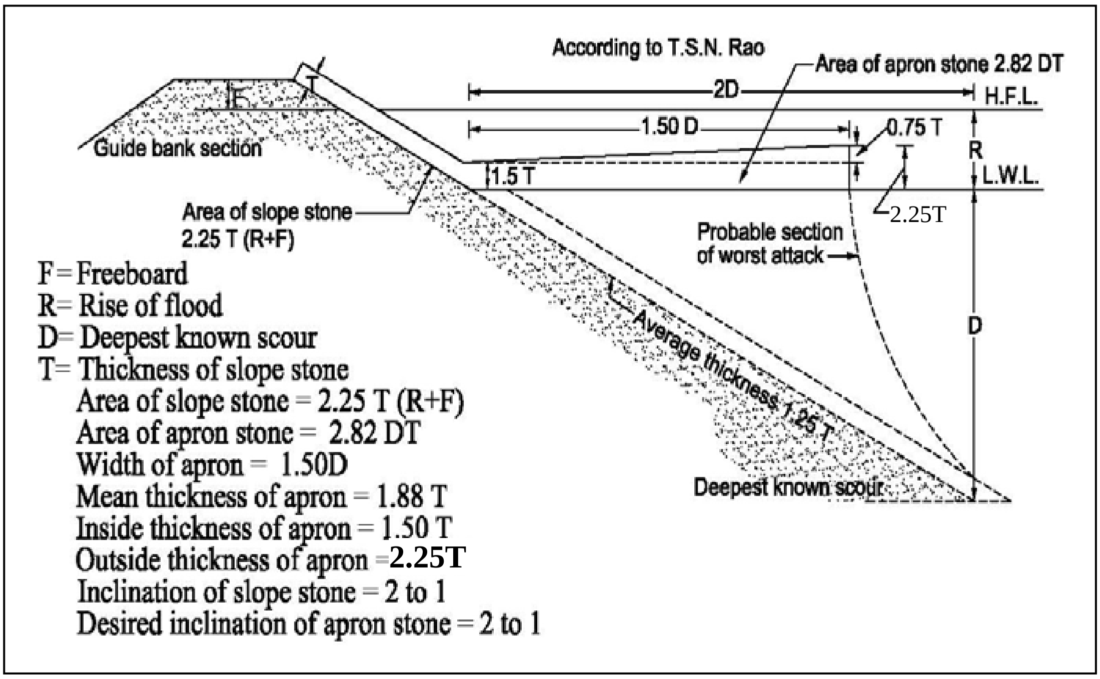


Figure 3.1: Shape of apron suggested by Rao(1946)

b) Size of Apron Stone

The required size of stone for launching apron may be the same as the size of stone in slope revetment considering stream velocity as governing factor.

c) Length of Launching Apron

The general practice as recommended by Inglish (1949) is to lay the apron over a length of 1.5D, where D is the design scour below the position of laying.

d) Quantity of Stone in Apron

Knowing the thickness of apron, the depth of maximum probable scour and the slope of the launched apron, the quantity of apron stone can be assessed. For dimensioning and estimating quantity of stone in apron Fig. 3.8 may be used.

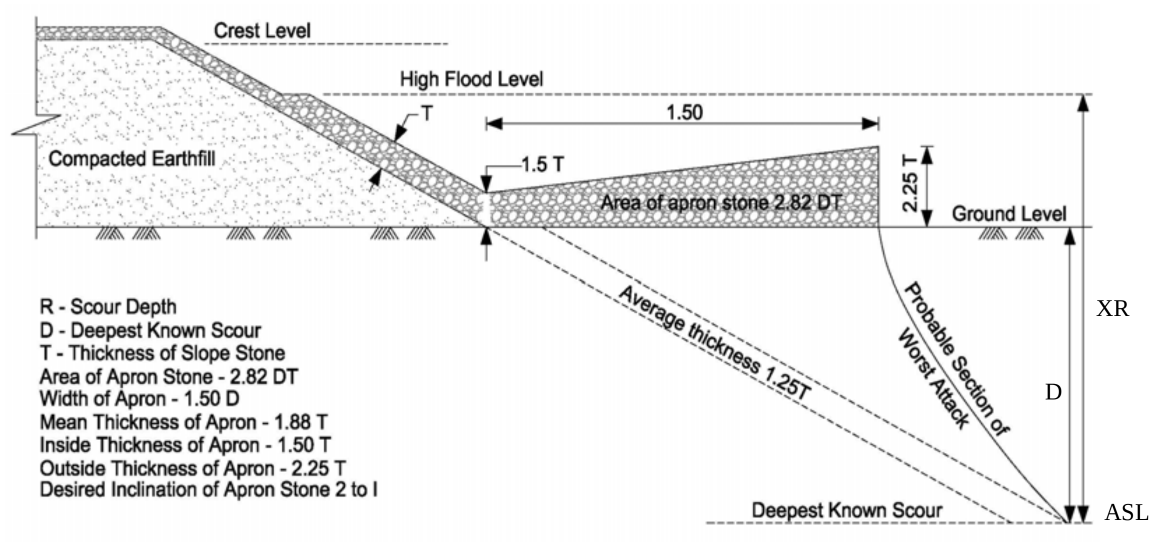


Figure.3.2 : Dimension of Launching Apron

CHAPTER FOUR

4. HYDRAULIC AND MORPHOLOGICAL CHARACTERISTICS OF JAMUNA RIVER

4.1 Origin and course of the Jamuna River

The **Jamuna River** is one of the three main rivers of Bangladesh and one of the largest in the world, with its basin covering areas in Tibet, China, India and Bangladesh. The Jamuna flows south, ending its independent existence as it joins the Padma River near Goalundo. Merged with the Padma, it meets the Meghna River near Chandpur. Its waters then flow into the Bay of Bengal as the Meghna River.

Actually Jamuna is the downstream course of the Brahmaputra which took place after the Earthquake and catastrophic Flood in 1787. Presently the Brahmaputra continues southeast from Bahadurabad (Dewanganj Upazila of Jamalpur district) as the Old Brahmaputra and the river between Bahadurabad and Aricha is the Jamuna, not Brahmaputra. It originates in the Chemayung-Dung glacier, approximately at 31°30'N and 82°0'E, some 145 km from Parkha, an important trade centre between lake Manassarowar and Mount Kailas. The Brahmaputra is known as the Dihang in Assam Himalayas before it comes into the Great Plains of Bengal. It enters Bangladesh through Kurigram district (at the border of Kurigram Sadar and Ulipur upazilas). The total length of the Tsangpo-Brahmaputra-Jamuna river up to its confluence with the Ganges is about 2,700 km. Within Bangladesh territory, Brahmaputra-Jamuna is 276 km long, of which Jamuna is 205 km.

The catchments of the mighty Brahmaputra-Jamuna river is about 5,83,000 sq km of which 293,000 sq km are in Tibet, 241,000 sq km in India and only 47,000 sq km within Bangladesh. The drainage area above Bahadurabad is 536,000 sq km. This is the widest river system in the country flowing north-south. There are gauges and discharges records for this river at Bahadurabad, where amount represents the flow entering Bangladesh plus those of the Dudhkumar, the Dharala and the Teesta, and minus those of the Old Brahmaputra and Bangali. The discharge during the rainy season is enormous, averaging 40,000 cumecs, by which measure it ranks with the Amazon, Congo, La Plata, Yangtze, Mississippi and Meghna as one of the seven largest rivers. The highest recorded flood was 98,600 cumecs in August 1988.

Average annual flow at Bahadurabad is estimated to be 501 million acre-feet. August has always been the month when widespread flooding has been most likely. Floods from May to July are usually due to the Brahmaputra-Jamuna and Meghna. From August to October due to the combined flows of those rivers and the Ganges. As a rule, the flow of the Brahmaputra-Jamuna is more erratic than that of the Ganges. The gradient of the Jamuna averages 1:11,850 which is slightly more than that of the Ganges. The Jamuna discharges a large volume of water and at the same time brings in huge amounts of Sediments. During the rainy season it brings down something like 1.2 million tons of sediment daily, and the annual silt runoff at Bahadurabad is estimated at 735 million tons. It has four major tributaries: the Dudhkumar, the Dharala, the Teesta and the Karatoya-Atrai system. The first three rivers are flashy in nature, rising from the steep catchments on the southern side of the Himalayas between Darjeeling in India, and Bhutan. Of all the distributaries, the Old Brahmaputra is the longest and was actually the course of the present Brahmaputra some 200 years ago.

4.2 Bank line shifting characteristics over the years

The satellite imageries of Jamuna river for last thirty years show that the river channel near the right bank is highly active and there is a strong trend of rightward shifting of the bankline. The average easting for right bank of the Jamuna river moved about 1.5 km to the west between the year 1973 and 1961. The bank line shifting of the study area from year 1973 to 1996 is shown in Figure 4.1. The unsteady movement of the Jamuna right bank particularly along the study area has also been observed in the recent years.

4.3 Geometry of the Jamuna River

The river's average depth is 395 feet (120 m) and maximum depth is 1,088 feet (332 m). The width of the river varies from 3 km to 18 km but the average width is about 10 km. In the rainy season the river is nowhere less than five kilometers broad. The river is in fact a multi-channel flow. Channels of many different sizes, from hundreds of meters to kilometers wide, and of different patterns including braiding, meandering and anastomosing pattern in the country. It is, through most of its course within Bangladesh, studded with islands (Chars) many of which are submerged during the rainy season and makes a single water channel. Thus, by breadth alone, this river qualifies as one of the largest in the world. The width/depth ratios for individual channels of the Brahmaputra vary from 50:1 to 500:1. The gradient of the river in Bangladesh is 0.000077, decreasing to 0.00005 near the confluence with the Ganges.

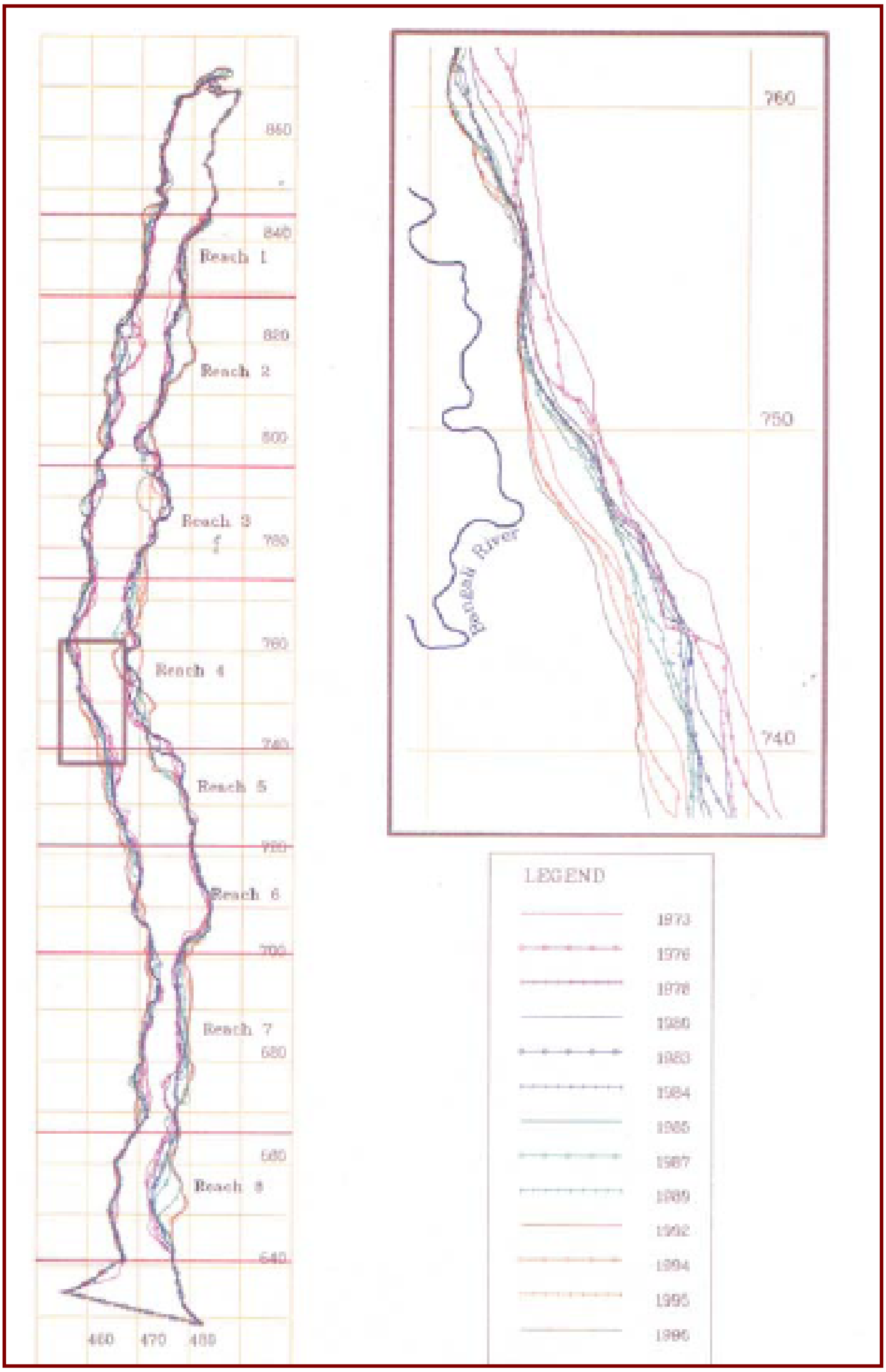


Figure 4.1: Right Banklines of Jamuna River from 1973 to 1996

4.4 Morphological characteristics of the Jamuna River

The Jamuna is a dynamic river of highly braided nature. The river has got multiple numbers of inter connecting channels. The direction of individual channel varies from +90 and -90 degrees where the larger channels show smaller deviations from the river axis. Numerous alluvial chars (permanent and moving) are the characteristics of the river. The presence of bars obstructs flow and scour occurs, either lateral erosion of banks on both sides of the bar, scour of the channels surrounding the bar, or both. This erosion enlarges the channel and reduces water levels.

The position of braids is likely to shift during floods, resulting in unexpected velocities, angle of attack and depths of flow. Lateral migration of braided streams takes place by lateral shift of a braid against the bank. The morphological behaviour of the Jamuna river near the study area could be understood from the plan form changes as shown in Figure 4.2. The slope of the river decreases in the downstream direction. Near Bahadurabad it is about 8 cm/km, while near the confluence with the Ganges near Aricha it is about 6 cm/km.

Within the braided belt of the Jamuna, there are lots of chars of different sizes. An assessment of the 1992 dry season Landsat image shows that the Jamuna contained a total of 56 large island chars, each longer than 3.5 km. There were an additional number of 226 small islands/ chars, varying in length between 0.35 and 3.5 km. This includes sandy areas as well as vegetated chars. In the Jamuna the period between 1973 and 2000, chars have consistently appeared in the reaches opposite to the Old Brahmaputra off-takes, north and east of Sirajganj and in the southernmost reach above the confluence with the Ganges. In entire Bangladesh during 1981 to 1993, a total of about 729,000 people were displaced by Riverbank Erosion. More than half of the displacement was along the Jamuna.

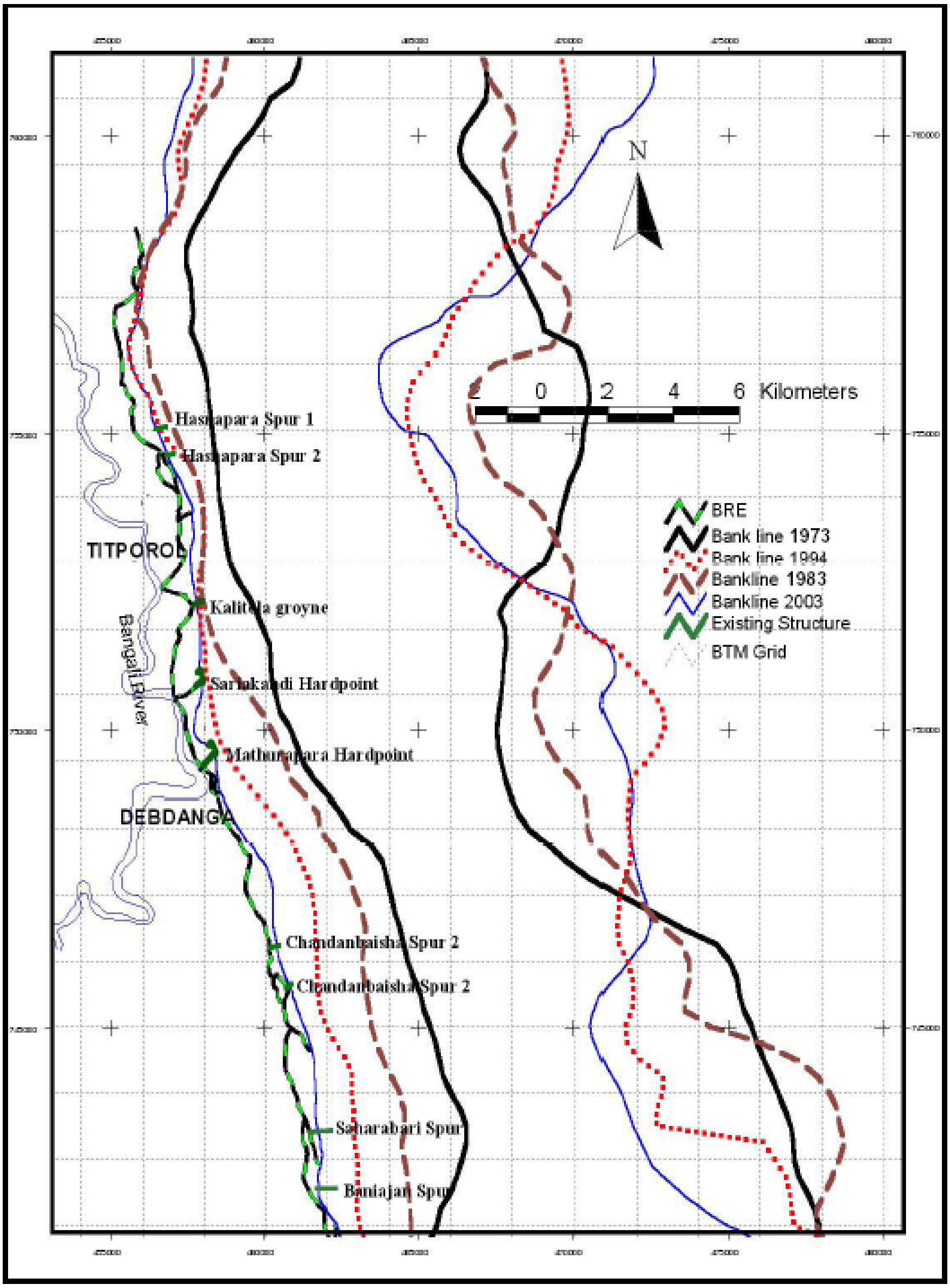


Figure 4.2: Historical Bankline and Planform Changes of Jamuna River near the study area (Titporol and Debdanga during 1973-2003

CHAPTER FIVE

5. PERFORMANCE ANALYSIS AND DISCUSSION

5.1 Study area

The study area is located along the right bank of the Jamuna river at Titporol and Debdanga under Sariakandi Upazila in the Bogra district. Titporol is situated at immediate upstream of the existing Kalitola groyne and Debdanga is at 1.0 km downstream of the existing Mathurapara Hard Point. The location of the study area is shown in Figure 5.1. During monsoon 2003, about 187 m of area at Titporol was washed away and that time the Brahmaputra Right Embankment (BRE) was only about 40m away from the Jamuna. If the bank erosion would continue like this then there would threat of unification of Bangali and Jamuna flow at this area. On the other hand, near Debdanga, the right anabranch of the Jamuna river was flowing along the river bank. During the same monsoon period there was a report of about 140m bank erosion at this area and the Jamuna was only 210m away from the Bangali. Under this situation BWDB implemented two protective works at Titporol and Debdanga in accordance with the recommendation of model study.

5.2 Protective works at Titporol and Debdanga

Based on the erosion mitigation measures recommended by IWM through modeling study in pre-monsoon 2004 (Technical notes, IWM, January 2004), BWDB implemented the protective works at Titporol and Debdanga along the Right Bank of the Jamuna during November 2004 to April 2005. Based on the design parameters supplied by IWM the detailed design was made for the protective works. The implemented protective works (2000 m revetment at Titporol from 200 m upstream of existing Kalitola Groyne and 1273 m revetment at Debdanga from 600 m downstream of existing Mathurapara Hard Point) are according to the Option 1 recommended by IWM. The implemented protective works are shown in Figures 5.2 and 5.3.

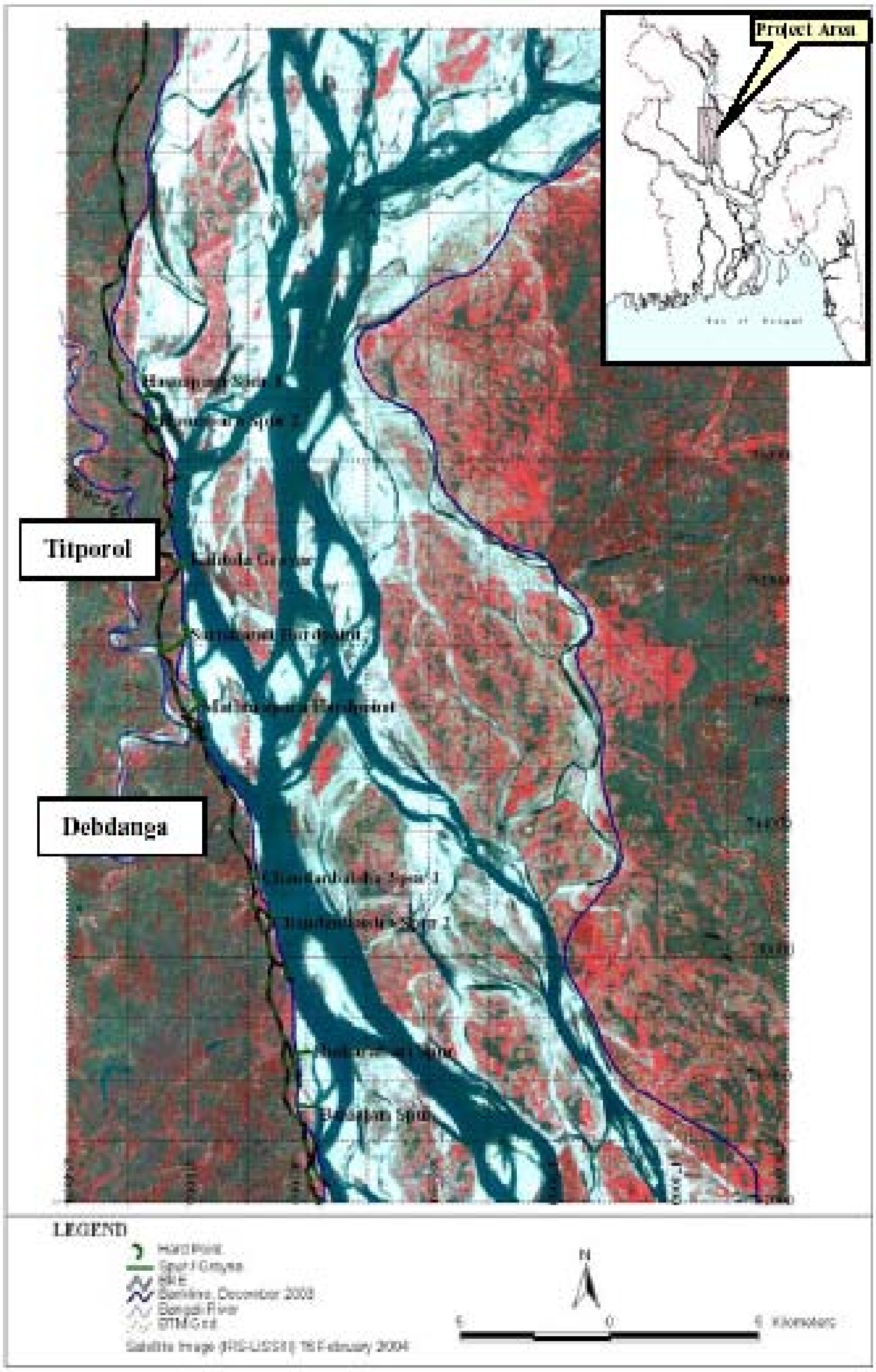


Figure 5.1: Study Area showing the channels of the Jamuna River

Figure 5.2: Titporol Revetment along the Right Bank of Jamuna River

Figure 5.3: Debdanga Revetment along the Right Bank of Jamuna River

5.3 Model studies for the protective works

BWDB commissioned IWM to carry out modelling study for planning and designing of bank protection work at Titporol and Debdanga in November 2004 through a formal contract signing. Beforehand, on request of BWDB, IWM submitted a Technical report in January 2004 containing the planning options for the crucial bank protection works at Titporol and Debdanga. This report was prepared based on the morphological model developed for Special O&M programme at Site B1 structures of RBPP using the measured 2004 pre-monsoon bathymetry data.

The sustainability of river training works largely depends on river response due to the intervention in terms of river bank erosion/accretion, channel and char development especially in the braided river. Hence, to predict such developments and to suggest appropriate protective measures three two dimensional morphological models were developed using data of 2004, 2005 and 2006. Two-dimensional mathematical modeling tool MIKE 21C of DHI Water and Environment has been employed for the modeling study. The sustainability of the implemented works largely depends on future development of rivers in terms of bank erosion/accretion, channel and char development etc. 2-dimensional modelling enables to predict such developments and to suggest appropriate protective measures in case of any adverse developments during the on-going implementation of the protective measures through adaptive and monitoring modelling. MIKE 21C operates on curvilinear computational grid, incorporates fully unsteady flow, bed load as well as suspended load via a non-equilibrium transport formulation taking into account both phase lag and time lag effects in adaptation of suspended load. The suspended load is calculated with the inclusion of the inertia of the sediment in suspension i.e. adaptation and the effect of the helical flow is accounted through profile function.

The following modelling study have been carried out regarding the implementation of Titporol and Debdanga revetments

5.3.1 Modelling for Devising Mitigation Measures (Pre-monsoon 2004)

A two-dimensional morphological model, covering the study area, was developed, validated and calibrated in connection with RBPP, Site B1 for monsoon 2003. The model was validated against 2003 monsoon hydrology. Dueing planning stage of the study in pre-monsoon 2004, this model was updated with the bathymetry measured by IWM in pre-monsoon 2004 under RBPP, Site B1 Project. In the updated model, morphological verification was done against pre-monsoon 2004 bathymetric data (hind cast). The model also included the proposed river training work to compare the impact of proposed river training works on existing water environment. The model was also employed to investigate the hydro-morphological condition under different application scenarios.

5.3.2 Modelling for Adaptive Measures and 2005 Monsoon Monitoring

The two-dimensional morphological model of Titporol-Debdanga in the Jamuna (developed during planning stage) was updated with the bathymetry of pre-monsoon 2005. The updating included generation of the computational grid with the river bank line data, updating the model topography, hydrodynamic validation and morphological verification of model against 2004 hydrological year. Such validation enhanced the confidence limit of the forecast model and provided basis for adapting the planning stage design considering updated bathymetry.

5.3.3 Modelling for Adaptive Measures and 2006 Monsoon Monitoring

The two-dimensional morphological model of Titporol-Debdanga in the Jamuna developed during implementation stage and updated and validated during 2005 monitoring, has been further updated with the bathymetry of pre-monsoon 2006. Similar updating and validation procedure and adaptive measures have been maintained for monsoon 2005. The bathymetry of Jamuna river for pre-monsoon 2006 used as initial condition for the operation and monitoring model for 2006 monsoon.

5.4 Design of the protective works

The design of protective works at Titporol and Debdanga was prepared by Design Circle-VI of BWDB in accordance with the findings and suggestions given by IWM. As mentioned earlier. Typical design of Titporol revetment is shown in Figure 5.4. The detailed design parameters and design data are provided in Table 5.1 and 5.2 respectively.

Table 5.1: Detailed design parameters

SL. No.	Parameter	Value
1	Total length of revetment	2000 m
2	Slope of slope pitching by CC Block	1:2
3	Length of Launching Apron	26.00 m
4	Volume of Launching Apron	52 m^3/m(excluding void)
5	Size of CC Block of slope pitching	40 cm x 40 cm x 40 cm
6	Average thickness of Launching Apron	2.00 m
7	Thickness of Geo-textile Filter	3.00 mm
8	Thickness of sand filter	100 mm

Table 5.2: Design data

SL. No.	Item/Parameter	Value
1	Lowest Water Level (LWL)	10.00 m, PWD
2	Highest Water Level (HWL)	19.07 m, PWD
3	Maximum discharge of Jamuna River	1,02,535.00 m^3/s
4	Design discharge (70% of maximum discharge)	71774.50 m^3/s
5	Silt factor (f)	0.70
6	Lowest bed level	6.64 m, PWD

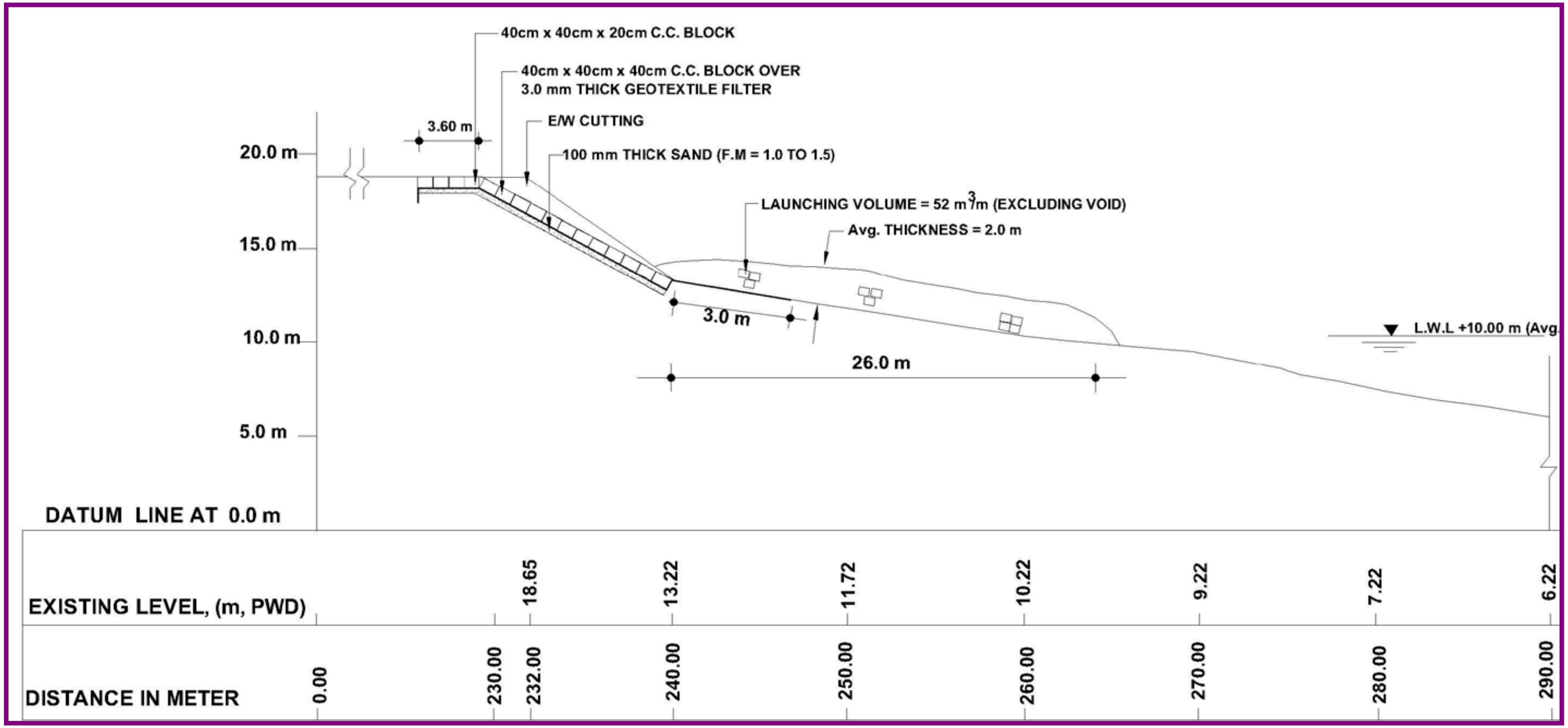


Figure 5.4: Typical Design Section of Titporol Revetment drawn at Km. 17.00 (C/S No. 20)

5.5 Performance of the protective works

From field investigation on 28 June 2005 by IWM it was found that the revetment work at Debdanga performed well during early flood after construction. Some portion of upstream revetment at Titporol, however, damaged in June 2005 and mitigation measures by dumping of earth filled synthetic bags were carried out to stop further collapsing of river bank. From BWDB field source it is seen that the Titporol revetment from M -60.00 to M 0.00, from M 0.00 to M 360.00, from M 1125.00 to M 1600.00 (total 895 m) was damaged. Typical failure is shown in Figure 5.5.

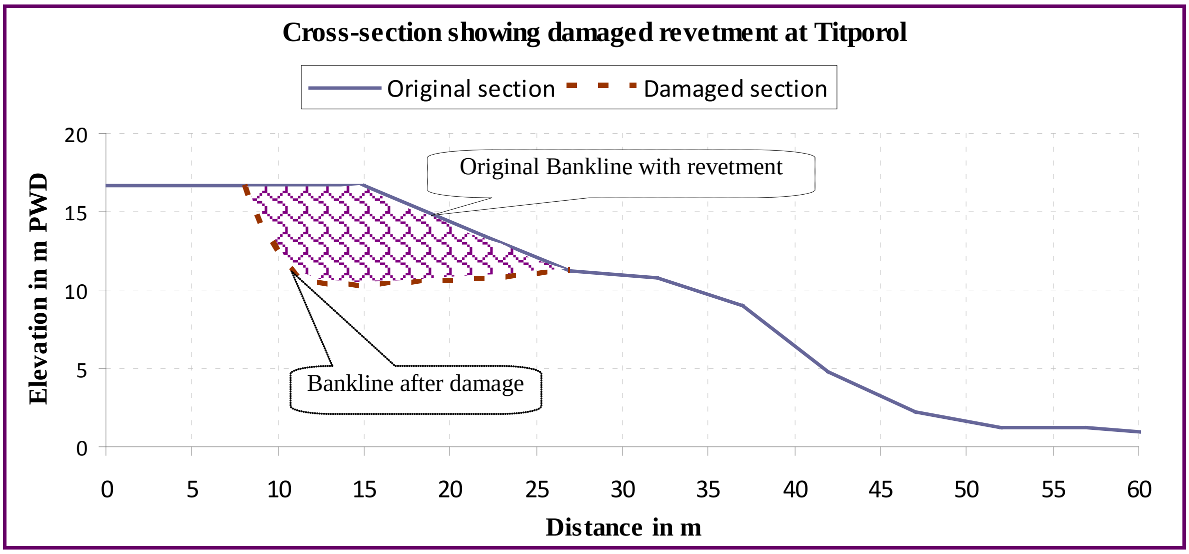
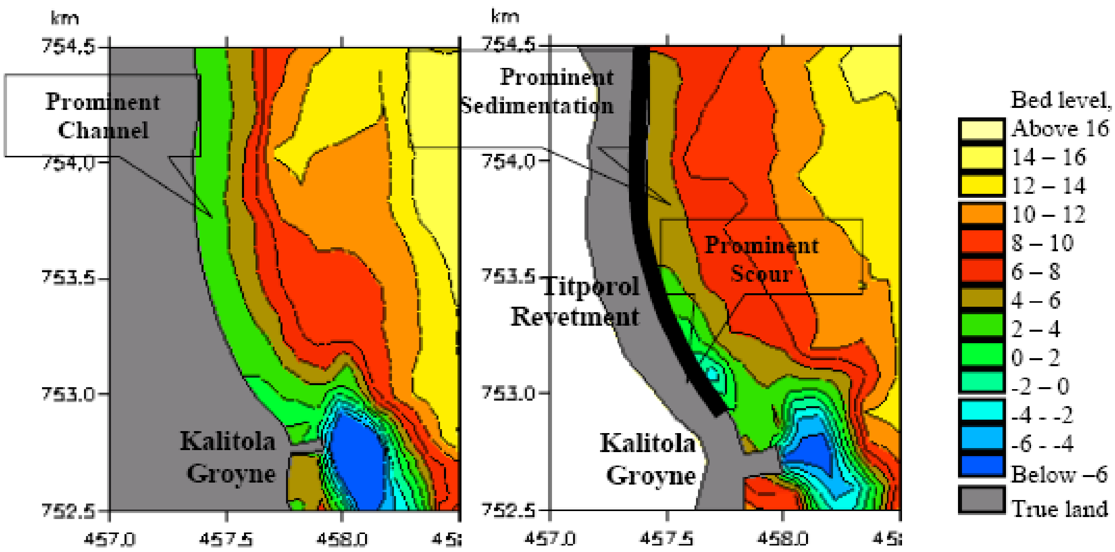


Figure 5.5: Cross-section showing typical bank failure of Titporol Revetment

From some surveyed cross-sections, taken by IWM after the repair works it was observed that the apron launched to lowest bed level of -3.00 m PWD while the same was initially laid at level of about +7.50 m PWD. The anticipated scour level was -13.92 m PWD.

In order to evaluate the performance of Titporol and Debdanga revetments, measured bathymetry/cross section data (surveyed by IWM) for pre- and post monsoon of 2005 were compared and shown in Figure 5.6 and 5.7. The near bank channel pattern along Titporol and Debdanga, near the newly constructed revetments, was changed after 2005 monsoon. Plan view of the measured bed level for pre- and post-monsoon 2005 shows that sedimentation occurred at upstream of Titporol revetment whereas scour occurred near downstream tip of the revetment. Near Debdanga revetment sedimentation occurred in front of the revetment within one monsoon of 2005. Model study also shows that if the revetment would not be constructed, then the bed scour would more aggravate the situation. This indicates the effectiveness of the revetment.

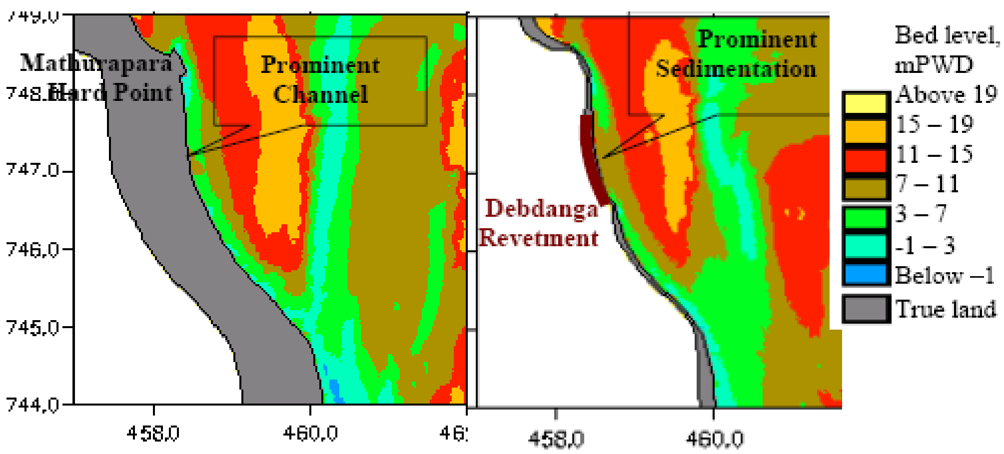
The vertical change of the channel bed after one monsoon (2005) near Titporol and Debdanga revetments were also investigated using the data collected by IWM. It is seen that near downstream tip of Titporol revetment, river bed erosion occurred due to monsoon of 2005. At Debdanga revetment, deposition occurred after 2005 monsoon indicating the effectiveness of the revetment at this reach.



Based on pre-monsoon 2005 data before construction of Titporol revetment

Based on pre-monsoon 2006 data after construction of Titporol revetment

Figure 5.6: Comparison of bed level in front of Titporol revetment before and after construction of the revetment indicating sedimentation near Titporol revetment



Based on pre-monsoon 2005 data before construction of Debdanga revetment

Based on pre-monsoon 2006 data after construction of Debdanga revetment

Figure 5.7: Comparison of bed level in front of Debdanga revetment before and after construction of the revetment indicating sedimentation near Depdanga revetment

Source: IWM 2005

5.6 Investigation of cause of failure of Titporol Revetment

The river bank protection works may be failed due to rapid change of hydro-morphological conditions of the river at or near the implemented structures and also by the failure of the riverbank /embankment itself due to sliding of soil mass along a curved surface. In order to find the probable cause of failure of the protective work at Titporol, investigation has been carried out through checking the adequacy of the design of the revetment, slope stability analysis and analysis of field condition during damage of revetment.

5.6.1 Checking Adequacy of Design

On the basis of collected field data and observation, necessary design parameters have been analyzed. C.C. block sizes were determined by applying commonly used formulas and those were compared with the sizes actually provided in the implemented revetments at Titporol and Debdanga. Adequacy of blocks in terms of size and weight has been assessed by following this procedure. Combining this with the conditions observed in field, performance of the bank protection works have been evaluated in terms of (i) effectiveness- whether the protective works implemented could effectively cease bank erosion, stabilize the slope, withstand flow slides and protect toe (ii) appropriateness- if the types of protection works were appropriate for the situation in consideration of river size, current, waves, etc.

Major design criteria to determine the size of the CC blocks are the resistance against probable maximum near bank velocity and wave action, while the depth of probable maximum scour governs the volume of launching apron materials. The maximum size of river bank soil material determines the type of geo-textile fabrics to be used as filter.

There are several formulas for evaluating the size of CC Blocks against near bank velocity and significant wave height. However, Neills' method and JMBA equations are preferred for estimating normal size of cc blocks, while Pilarczyk equation is used against wave action for the purpose. The following equations are used for checking the design. The detail computation is given in appendix-A.

A. Determination of size of Revetment: Design against current attack

a) Size against velocity, Neil's Method

Diameter of Stone, D = $0.034\ V^2$

where, D is the diameter of stone in m and
V is the velocity of flow in m/sec

b) Using JMBA Equation (This equation has been developed for CC Block)

$$D_n = \frac{0.7V^2}{2(Ss-1)g} \times \frac{2}{\log(6h/d)^2} \times \frac{1}{[1-(\sin\alpha/\sin\theta)^2]^{0.5}}$$

where, • = angle of repose of cc block

c) Using Pilarczyk (1990/2000) equation

Size (dia of stone) of revetment cover layer material, D_n $= \dfrac{0.035\hat{u}^2}{\Delta_m 2g} \dfrac{\varphi_{sc} K_\tau K_h}{K_s \psi_{cr}}$

where, h = Water Depth

•$_s$ = Density of protection material

•$_w$ = Density of water

Ss =Specific gravity

• m = relative density of submerged material

g = acceleration due to gravity

•$_{sc}$ = Stability factor for current

•$_{cr}$ = Critical shield stress parameter
u = Average flow velocity

K• = Turbulence factor

• = Slope angle of bank

• = Angle of repose of protection material

Ks = Slope factor = • [(1- (sin • / sin •)2]

B. Determination of size of revetment against wave attack

<u>Pilarczyk equation</u>

Size of revetment material, D_n = $\dfrac{H_s \xi_z^{\,b}}{\Lambda_m \psi_u \phi_{sw} \cos\alpha}$

where,

Significant wave height, Hs

Stability factor for incipient motion, $\bullet_{sw}$

Stability upgrading factor, $\bullet_{u}$

Mean wave period, Tm

Wave similarity parameter, •z = tan• (1.25Tm/• Hs)

Wave structure interaction coefficient, b

5.6.2 Slope stability analysis

An earthen embankment usually fails, because of the sliding of a large soil mass along a curved surface. It has been observed that the surface of slip is usually close to cylindrical, i.e. an arc of a circle. The method which is used for computing stability of slope of an embankment is known as Slip Circle Analysis. The minimum factor of safety against slip circle failure is considered as 1.5. The computation of slip circle analysis has been made using computer software “XSTABL” which shows **a factor of safety less than 1.5 (1.13).** The geotechnical data used for the analysis are moist, saturated, cohesion(c) and angle of internal fraction. The details computation of slope stability is given in Appendix-B.

5.6.3 Field data analysis and investigation

Cross-sectional data of the study area surveyed by BWDB has been collected from field office and analysis of the data is carried out during the course of this study. The data was surveyed during post-monsoon of 2005 (after failure of revetment). From the analysis of these cross-sections it is observed that most of the damage occurred at the slope of the newly built slope for pitching above low water level. The launching apron functioned well during first monsoon after implementation of the revetment as shown in Figure 5.5.

It has been reported by the field engineers of BWDB that there were a good numbers of ditches/ponds within 50m of the revetment at Titporol. From the analysis it is revealed that the damage occurred during last week of June 2005 due to low shear strength of soil and in presence of pore water pressure. It is apprehended that for lack of free drainage, pore water pressure developed behind the geo-textile and this resulted failure of bank along a curved surface. At the damaged portion, subsoil water might have been drained from underground source or from the existing ponds behind the revetment as was found at the site.

From analysis of morphological data surveyed by IWM it has been seen that about 120m to 150m bank erosion occurred along the right bank of Jamuna River at Antarpara at the upstream of Titporol revetment. This morphological change may be another reason for the damage of the upstream tip of the revetment during 2005 monsoon.

5.7 Comparative Study of the Protective Works

5.7.1 Introduction

In order to find the probable cause of failure of the revetment at Titporol a comparative study of the implemented revetment works at Debdanga and Titporol was as made in terms of design and the performance during the first monsoon they faced in 2005. The observations made during the study through data collection and analysis, field visits and interaction with field officials, review of the design of the corresponding works are discussed in this chapter. The different design and implemented parameters of the revetments are compared and shown in Tables 5.3 and 5.4.

Table 5.3 : Data considered for the design of bank protection work

No	Data	At Titporol	Remarks
1	Highest Water Level (HWL)	19.07 m (PWD)	Same data was used for Debdanga protective work
2	Lowest Water Level (LWL)	10.00 m (PWD)	
3	Design discharge	71774.50 m^3/s	
4	Design Velocity (from model study)	3.00 m/s	
5	Silt factor, f	0.80	
6	Design Bank Slope above LWL	1:2	
7	Wind Fetch Length	10.0 km	
8	Wind Velocity	20.00 m/sec	
9	Significant wave height	0.90 m	
10	Wave period	3.40 sec	
11	Duration of Wind	1.75 hour	

Table 5.4 : Comparison of salient designed and implemented features of revetment

No	Data	At Titporol	Remarks
1	Thickness of pitching above LWL	1.10 m	Same design features are provided for Debdanga protective work
2	Launching volume	52.00 m^3/m	
3	Avg. thickness of Launching Apron	2.00 m	
4	Length of Launching Apron	26.00 m	
5	Size of pitching block	40cmx40cmx40cm	
6	Size of launching block	40cmx40cmx40cm-30% 45cmx45cmx45cm-50% 50cmx50cmx50cm-20%	
7	Calculated Scour level	-13.92 m PWD	

For the purpose of evaluation, the present study compared the different parameters measured in pre-monsoon 2005 (without project condition) and post-monsoon 2005 (with project condition) along the right bank of Jamuna River near Titporol and Debdanga areas. Based on the measured data, following are the assessments on the performance of the Titporol and Debdanga revetments during monsoon 2005.

1. Both revetments, at Titporol and Debdanga, were fully capable to stop the bank erosion at the right bank of Jamuna river within the area. It is seen from the findings of model study that if the revetments were not constructed (hypothetical situation),

then the bank would more aggravate the situation. This indicates the proper functioning of the revetments.

2. About 120m to 150m bank erosion occurred along the right bank of Jamuna river at Antarpara at upstream of Titporol revetment. As a result, the upstream tip of Titporol revetment was damaged during monsoon 2005. But no such things occurred for the Debdanga revetment.
3. Prominent sedimentation occurred in front of Debdanga revetment whereas significant sedimentation occurred near the upstream tip of Titporol revetment.
4. Slip circle failure occurred at different locations of the Titporol revetment. But no such damage occurred at the Debdanga revetment.
5. There were a good numbers of ponds/depressions behind the Titporol revetment, but there were no such features behind the Debdanga revetment.

5.8 Discussion of Cause of Failure of Protective Works at Titporol

Design of the revetment has been reviewed using the standard procedure mentioned in the Design Manual of BWDB and have been found satisfactory. The detail design calculation is provided in the Appendix-A.

The factor of safety against slip circle failure has been found to be 1.13 which is less than the minimum allowable factor of safety of 1.5. The detail slope stability computation is given in Appendix-B.

From the analysis of field condition it is revealed that the damage that occurred during June 2005 due to low shear strength of soil and in presence of pore water pressure. It is apprehended that for lack of free drainage, pore water pressure developed behind the geo-textile and this resulted failure of bank slope. At the damaged portion, subsoil water might have been drained from underground source or from the existing ponds behind the revetment as was found at the site during field visit.

CHAPTER SIX

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The following conclusions are drawn from this work.

i) Field visits have been made to the sites to investigate the performance of the revetment works and to determine the probable cause of failure at some locations of Titporol revetment;

ii) Both revetments, at Titporol and Debdanga, seems to be fully capable to stop the bank erosion at the right bank of Jamuna river within the area;

iii) Design calculation shows that calculated same amount of dumping material was placed for both the sites;

iv) Debdanga was found more effective than that of Titporol;

v) Slip circle failure occurred at different locations of the Titporol revetment. But no such damage occurred at the Debdanga revetment;

vi) Damage occurred due to low shear strength of soil and in presence of pore water pressure and

vii) Due to lack of free drainage, pore water pressure developed behind the geo-textile and this resulted failure of bank slope

6.2 Recommendations for Future Work

i) This study has been done on protection works implemented at Titporol and Debdanga along the right bank of the Jamuna River. However similar comparative study can be carried out for other similar locations on the river.

ii) More field data can be collected in order to carry out such comparative analysis.

iii) Available low cost bank protection method can be used for future comparison of traditional revetment works.

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Appendix-A : Sample Design Computation

Peak discharge of Jamuna river at Bahadurabad = 1,02,535 m^3/s (25 year flood as found from the frequency analysis shown Appendix-C).

Design discharge, Qd = 70% of Peak Discharge = 71,774.50 m^3/s

Design of Thickness of Rip-rap:

i) Using Inglish Formula

Thickness, t = $0.06Q^{1/3}$ m
= 0.06 x (71774.50)1/3 m
= 2.50 m

ii) According to Spring

For medium bed material, provided thickness for river slopes per mile is 31 inch (• 0.80 m).

iii) According to Gale

In accordance with Gale's, for a river having discharge from 1.50 million to 2.50 million cusec, the thickness, t = 3'-6" = 3.50 feet = 1.066 m.

From the above analysis the reasonable thickness of the rip-rap has been found to be 1.00 m.

Design of Launching Volume:

Calculation of Scour depth:

Lacy's regime normal scour depth, $R = 0.47\left(\frac{Qd}{f}\right)^{1/3}$

$$= 0.47\left(\frac{71774.50}{0.80}\right)^{1/3} \text{ m}$$

= 21.00 m

Maximum scour depth = 1.50 x Normal scour depth (XR)
= 1.50 x 21.00 m (X=1.5 for moderate bend)
= 31.500 m

Anticipated scour level, ASL = HFL – XR
= 19.07 – 31.50
= -12.43 m PWD

Lowest bed level = 6.64 m PWD higher than -12.43 m, PWD

Therefore, Depth of scour, D = LWL – ASL
= 10.00 – (-12.43) m
= 22.43 m

Launching volume, V = $\sqrt{1.5^2 + 1^2}$ x 1.25t x D (For CC Block)
= $\sqrt{1.5^2 + 1^2}$ x 1.25 x 1.00 x 22.43 m^3/m
= 1.80 x 1.25 x 1.00 x 22.43 m^3/m
= 50.47 m^3/m (provided 52 m^3/m)

Hence, O.k. regarding launching volume.

<u>Checking Size of CC Blocks</u>

1. Determination of size of Revetment: Design against current attack

Revetment material : CC Block, single layer, hand place
Unit weight of CC block, • s = 2250 kg/m3
Unit weight of water, • w = 1000 kg/m3
Average velocity of flow, V = 3.00 m/sec

a) Size against velocity, Neil's Method
Diameter of Stone, D = 0.034 V^2
D = $0.034V^2$
= 0.034 x 3^2 m
= 0.306 m

Equivalent cube dimension = 247 mm

b) Using JMBA Equation (This equation has been developed for CC Block)

$$D_n = \frac{0.7V^2}{2(Ss-1)g} \times \frac{2}{\log(6h/d)^2} \times \frac{1}{[1-(\sin\alpha/\sin\theta)^2]^{0.5}}$$

H = water depth = 9.07 m

Slope angle of bank, • = 26.57° = 0.464 radians

Angle of repose of protection material, • = 40° = 0.698 radians
h/d = h/Dn = 9.07

By trial and error,

when we assume Dn = 0.1377 m, then calculated Dn = 0.2569 x 0.385086 x 1.39219 = 0.138 m

Therefore calculated Dn = 138 mm

c) Pilarczyk (1990/200) equation:

Size (dia of stone) of revetment cover layer material, D_n $= \dfrac{0.035\hat{u}^2}{\Delta_m 2g} \dfrac{\varphi_{sc} K_\tau K_h}{K_s \psi_{cr}}$

h = Water Depth = (HWL - LWL) = 9.07 m
• s = Density of protection material = 2250 Kg/m3
• w = Density of water = 1000 Kg/m3
Ss =Specific gravity = 2.25
• m = relative density of submerged material = 1.25
g = accleration due to gravity = 9.81 m/sec2
• sc = Stability factor for current = 0.65
• cr = Critical shield stress parameter = 0.05
Average flow velocity, u = 3.00 m/sec
K• = Turbulance factor = 1.0
• = Slope angle of bank = 26.56° = 0.464 rad
• = Angle of repose of protection material = 40° = 0.698 rad
Ks = Slope factor = • [(1- (sin • / sin •)2] = 0.718

Calculation of Kh (depth factor):
[Kr = Dn, for block mats]

For non developed velocity profile
Kh = Depth factor = [(h/Kr)+1)] -0.2 = 0.399

For developed velocity profile
Kh = depth factor = 2.[log (12h/kr)] -2 = 0.212

Very rough flow (h / kr < 5)
Kh = depth factor = N/A because h/d>5

Say design depth factor, kh = 0.399

$$D_n = \frac{0.035\hat{u}^2}{\Delta_m 2g} \frac{\varphi_{sc} K_\tau K_h}{K_s \psi_{cr}} = \frac{0.0818}{0.8808}$$
$$= 0.0928 \text{ m}$$

By trial and error, when assume Dn = 0.0930, then calculated Dn becomes 0.0928 m
Thus required Dn = 0.093 m = 93 mm

Required size of revetment material single unit (CC block) under current attack has been found to be 306 mm considering maximum of the above three values.

Size provided in implementation of revetment = 400 mm, hence the design is O.k under current attack.

2. Determination of size of Revetment: Design against wave attack

Size of revetment material, D_n = $\dfrac{H_s \xi_z^{\;b}}{\Lambda_m \psi_u \phi_{sw} \cos\alpha}$

Significant wave height, Hs = 0.90 m
Unit weight of revetment material, • s = 2250 Kg/m3
Unit weight of water, • w = 1000 Kg/m3
Relative density of submerged material, • m = 1.25
Acceleration due to gravity, g = 9.81 m/sec2
Stability factor for incipient motion,• sw = 2.25
Stability upgrading factor, • u = 1.50
Slope angle of bank, • = 26.57° = 0.464 rad
Mean wave period, Tm = 3.40 sec
Wave similarity parameter, • z=tan• (1.25Tm/• Hs) = 2.24 (<= 3, O.k.)
Wave structure interaction coefficient, b = 0.67

Therefore, size of Revetment material, Dn = **Error! Bookmark not defined.**

$$\frac{H_s \xi_z^{\;b}}{\Lambda_m \psi_u \phi_{sw} \cos\alpha} = \frac{1.5449}{3.773364}$$

= 0.4094 m
• 400 mm

Size provided in implementation of revetment = 400 mm

Hence, the design is Ok under wave attack !!!

Appendix-B : Slope Stability Computation

```
*****************************************
*                XSTABL                 *
*                                       *
*    Slope Stability Analysis using     *
*   Simplified BISHOP or JANBU methods  *
*                                       *
*          Copyright (C) 1991           *
*   Interactive Software Designs, Inc.  *
*         All Rights Reserved           *
*                                       *
*                                       *
*              GEOSYNTEC                *
*         Norcross,  GA 30093           *
*                                       *
*  Ver. 4.01                     1055   *
*****************************************
```

SEGMENT BOUNDARY COORDINATES

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.00	33.00	51.00	58.00	1
2	51.00	58.00	84.00	58.00	1
3	84.00	58.00	182.00	46.00	1
4	182.00	46.00	190.00	46.00	1
5	190.00	46.00	210.00	55.00	0

ISOTROPIC Soil Parameters

1 type(s) of soil

Soil Unit No.	Unit Weight Moist (pcf)	Sat. pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Constant (psf)	Water Surface No.
1	110.0	120.0	50.0	20.0	.000	.0	0

1 Water surface(s) have been specified

Unit weight of water =   62.40 (pcf)

Water Surface No. 1 specified by 2 coordinate points
****
PHREATIC SURFACE,
****

Point No.	x-water (ft)	y-water (ft)
1	12.00	38.80
2	153.00	48.00

A critical failure surface searching method, using a random technique for generating CIRCULAR surfaces has been specified.

100 trial surfaces will be generated and analyzed.

10 Surfaces initiate from each of 10 points equally spaced along the ground surface between x = 12.00 ft and x = 22.00 ft

Each surface terminates between   x =   51.00 ft and   x =   60.00 ft

Unless further limitations were imposed, the minimum elevation at which a surface extends is y =   .00 ft

* * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * * *

5.00 ft line segments define each trial failure surface.

ANGULAR RESTRICTIONS:

The first segment of each failure surface will be inclined within the angular range defined by :

Lower angular limit :=   -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

Factors of safety have been calculated by the MODIFIED BISHOP METHOD.

The most critical circular failure surface is specified by 11 coordinate points and the critical surfaces are shown if Figure B-1.

Point No.	x-surf (ft)	y-surf (ft)
1	12.00	38.88
2	17.00	38.83
3	21.98	39.31
4	26.88	40.31
5	31.64	41.82
6	36.22	43.83
7	40.56	46.30
8	44.62	49.23
9	48.35	52.56
10	51.70	56.27
11	52.95	58.00

Modified BISHOP FOS = 1.129<1.5

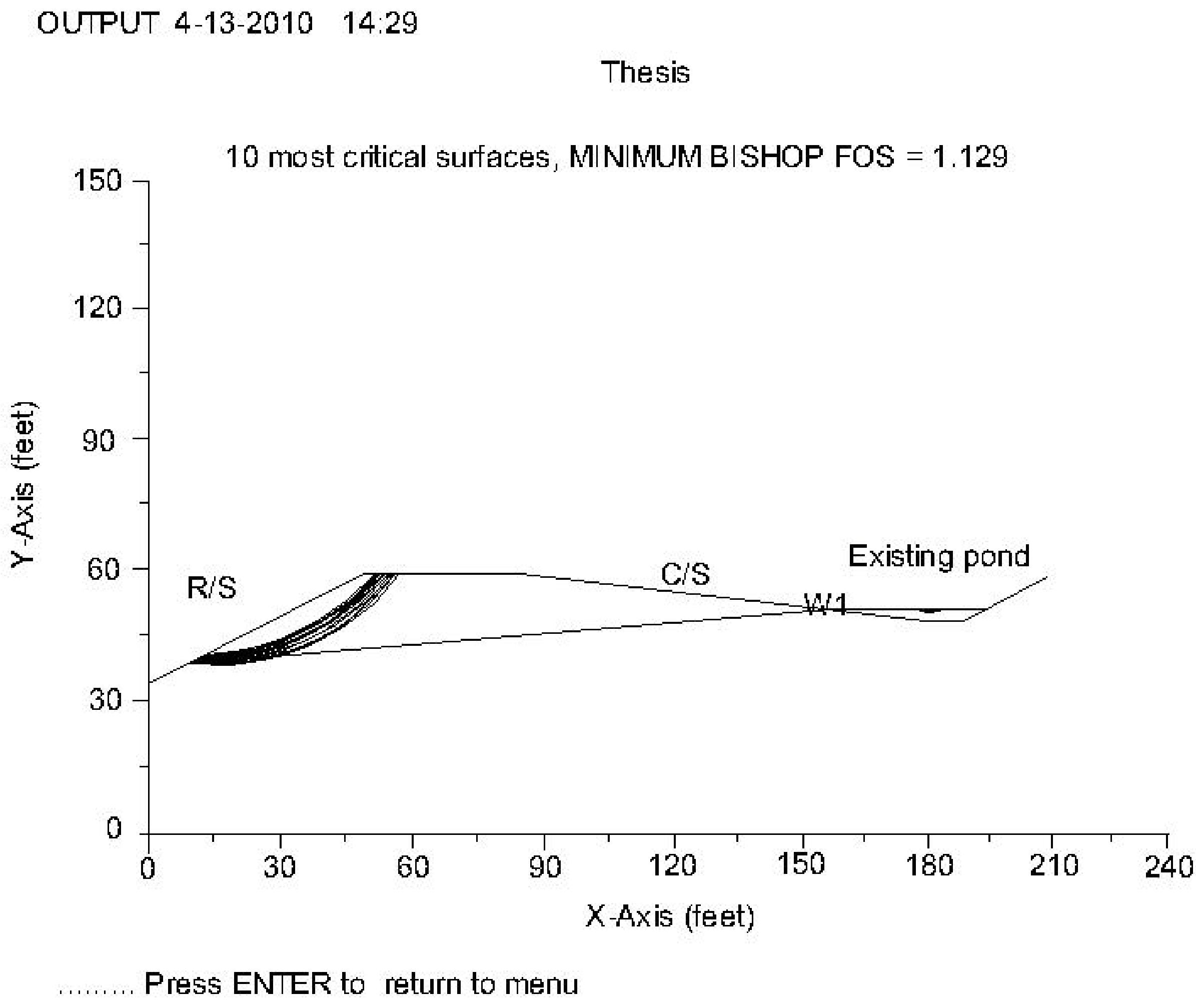

Figure B-1: X-Section of riverbank showing Critical slip circles

The following is a summary of the TEN most critical surfaces.

Surface	FOS (BISHOP)	Circle Center x-coord	Circle Center y-coord	Radius	Initial x-coord	Terminal x-coord	Driving Moment
1.	1.129	14.99	86.07	47.28	12.00	52.95	4.052E+05
2.	1.136	13.95	96.50	57.65	12.00	56.84	5.723E+05
3.	1.151	16.26	94.29	55.57	12.00	58.27	6.227E+05
4.	1.155	17.08	93.08	53.80	13.11	57.85	5.607E+05
5.	1.155	7.21	103.27	64.57	12.00	53.20	4.325E+05
6.	1.158	10.59	108.17	68.79	13.11	57.62	5.959E+05
7.	1.159	17.49	92.37	53.77	12.00	58.77	6.348E+05
8.	1.160	20.37	76.99	39.02	12.00	54.44	4.434E+05
9.	1.161	20.86	78.35	38.24	15.33	53.17	3.156E+05
10.	1.167	19.51	78.50	38.21	15.33	51.66	2.719E+05

- * * END OF FILE * * *

Appendix-C : Flood Frequency Analysis

Observed Data:

Year	Value (m^3/s)
1976	61700
1977	67300
1981	34200
1982	53500
1983	50900
1984	75700
1985	61900
1986	42000
1987	67200
1988	68700
1989	70500
1990	64400
1991	61500
1992	65600
1993	59100
1994	39100
1995	84200
1996	83485
1997	79270
1998	102535
1999	61915
2000	69320
2001	49230
2002	69728
2003	65684
2004	96106
2005	58767
2006	47666
2007	30329
2008	62379
2009	86939

Sample size = 31

Q_{mean} = 64221

Standard deviation,• = 16650.6

y_n = 0.53702

$•_n$ = 1.11558

Recurrence Interval, T	X_T	y	Frequency Factor, K	K•	Q_T (m^3/sec)
2	-0.521	0.367	-0.15	-2541.1888	61679.85
2.33	-0.613	0.579	0.04	624.710937	64845.75
5	-1.014	1.500	0.86	14378.815	78599.85
10	-1.340	2.251	1.54	25581.3298	89802.37
20	-1.652	2.971	2.18	36327.051	100548.09
25	-1.751	3.199	2.39	39735.737	103956.77
50	-2.057	3.903	3.02	50236.2848	114457.32
100	-2.360	4.601	3.64	60659.2982	124880.33
200	-2.662	5.297	4.27	71044.2798	135265.32
1000	-3.362	6.909	5.71	95100.1907	159321.23