

**A Physical Model Study On Bank Protection By Revetment In A Reach Of
The Old Brahmaputra**

A Thesis

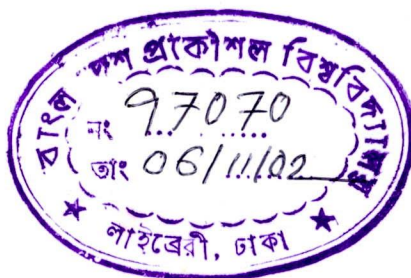
by

Shoma Tanzeeba

In partial fulfilment of the requirements for the degree

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CERTIFICATE OF APPROVAL

The thesis entitled "*A Physical Model Study on Bank Protection by Revetment in a Reach of the Old Brahmaputra*" submitted by Shoma Tanzeeba, Roll No.: 9516045 (F), Session: 1994-95-96 has been accepted as satisfactory in partial fulfillment of the requirement for the degree of Master of Science in Engineering in Water Resources Engineering on September 25, 2002.

BOARD OF EXAMINERS



(Dr. M. Monowar Hossain)

Professor

Department of Water Resources Engineering

BUET, Dhaka 1000, Bangladesh.

(Supervisor)

Chairman



(Dr. M. K. Alam)

Professor

Department of Water Resources Engineering

BUET, Dhaka 1000, Bangladesh.

Member



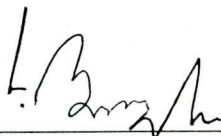
(Dr. M. A. Matin)

Professor and Head

Department of Water Resources Engineering

BUET, Dhaka 1000, Bangladesh.

Member



(Mr. Emaduddin Ahmad)

Executive Director

Institute of Water Modelling

Dhaka 1206, Bangladesh.

Member (External)

CANDIDATES DECLARATION

It is hereby declared that this thesis or any part of it has not been submitted elsewhere for the award of any degree or diploma.

Shoma Tanzeeba

(Shoma Tanzeeba)

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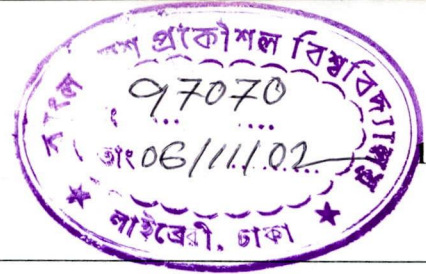
ABSTRACT

Bangladesh is one of the largest deltas in the world, which has been formed mainly with sediment deposited by the three mighty rivers namely the Ganges, the Jamuna and the Meghna. These alluvial rivers are continuously shifting their positions and changing shapes as a consequence of hydraulic and geomorphic forces acting on its bed and banks. The nature of this bank erosion and channel shifting problems demands intensive and careful long-term study both by physical (scale model) and mathematical models. For long term and short term hydraulic research activities including morphological studies on alluvial channels, a physical model facility has been developed the Department Water Resources Engineering, Bangladesh University of Engineering and Technology. This setup covers approximately 61 m length and 11.5 m width. It is equipped with 45.73m long and 10.67m wide sand bed, storage pools for water supply, upstream and downstream reservoirs, sedimentation tank, pumps, recirculating canal, measuring devices like weirs, water level recorders, point gauges, side rails and carriages etc. This physical model has been used for the study on bank protection by revetment.

In general, revetment structures are popular as bank protection work in Bangladesh. The different types of revetment, which are mainly used here, are cubical concrete blocks and loose stones. In the present study the performance of hexagonal block for bank protection work was evaluated and was compared with cubical blocks. Previous application of hexagonal block in case of wave attack has shown that it is very effective and less costly as a protection work. In this research work, applicability of the hexagonal block in case of current attack and its performance has been evaluated.

With this view, a scale model of an erosion prone reach of the Old Brahmaputra river has been developed. Then the performance of these two types of blocks i.e. hexagonal blocks and cubical blocks have been evaluated by placing them in the eroded bank keeping same condition for both of the blocks in different scenarios. The variables that are considered for the performance evaluation are different bank slopes (2H:1V and 3H:1V) and different values of velocity ranges. It has been

revealed from the study that hexagonal blocks required 50% less amount of material as compared to cubical blocks of equivalent weight for the selected bank protection area. Inspection of the bed profile showed that hexagonal shaped revetment structure suffered less damage than the cubical shapes for both the bank slope conditions. From side slope stability coefficient it can be concluded that flatter side slopes gives more stability for both the cases compared to higher side slope which is close to the angle of repose of the material. On the other hand, comparison between cubical and hexagonal type of revetments on same side slope gives better stability for hexagonal type revetment blocks. Other factors such as, time required for placement of these two kinds of blocks show that cubical blocks require 40% more time than as compared to hexagonal block due to their smaller shape and size.



CHAPTER ONE

1. INTRODUCTION

1.1 General

Bangladesh is a riverine country located in a deltaic region. Besides the three mighty rivers the Ganges, the Brahmaputra-Jamuna and the Meghna there are 259 rivers flowing throughout the country with a total length of about 24000 km. The behaviour of river, river system, estuaries and coastal areas are very erratic and complex. The nature of the bank erosion and channel shifting problems of these rivers demands intensive and careful long-term study both by physical (scale model) and mathematical models. Physical modelling is an advanced technique to solve the complicated river problems. If the model is designed and verified properly, then it can produce excellent results, which can be used successfully in the prototype.

The design of physical model is a technique, which deals with a lot of limitations and possibilities to attain a good solution. Comprehensive thinking about the relative importance of the accuracy of the model test results and the cost involved in the modelling is also essential. As the design of hydraulic river model is completely based on the trial and error approach, through more trial and optimization it is possible to get better solution.

For academic purpose physical model facilities including a multipurpose sand bed that can be used for long term and short term hydraulic research activities on alluvial channels is of utmost importance. In Bangladesh, studies on Bank Protection and River Training Works (RTW) have been conducted mainly for specific field purpose. River Research Institute (RRI) conducted physical model study of Jamuna river at the Jamuna Multipurpose Bridge Project (JMBP) area, at RRI, Faridpur, to solve the problems which may arise during the construction and maintenance phases against the hydrodynamic and morphological changes of the river around the bridge site (RRI, 1997). The model was constructed with a distortion of 3.5 considering horizontal scale of 1:280 and vertical scale of 1:80. Model study of Jamuna Bank Protection with revetment and River Training Pilot project at Bahadurabad was also

conducted by RRI, Faridpur (Yusuf, 1995). Several other studies on bank protection works were also conducted by RRI like Model study on Bank Protection of Sangu River at Dohazari, Meghna River Bank Protection Studies at Bhairab, Chandpur, Eklashpur, Brahmaputra River Training studies on Revetment stability and falling apron, Model study for Bank Protection at Chapra and Badanpur from erosion of River Gorai etc. (Yusuf, 1995). Also, Hossain and Khan have conducted experimental studies on groins as river bank protection work (Hossain, 1981, Hossain, 1987 and Khan, 1983). Both the studies were conducted in the Sand Bed of Hydraulic Laboratory, Department of Water Resources Engineering, BUET. But none of the above studies include the revetment structure like Hexagonal Armour Unit that are found to be very effective and less costly in Australia (Brown, 1983) in coastal region.

1.2 Objectives and Scope of the Study

Objectives with Specific Aims:

1. To participate in the design and develop of the physical model facilities including a multipurpose sand bed which will be used for long term and short term hydraulic research activities on alluvial channels in WRE, BUET.
2. To calibrate the physical model for which several test runs will be conducted.
3. To evaluate the performance of a revetment structure like Hexagonal Armour Unit for the river bank protection, a small scale model will be developed for different flow ranges and bank slope conditions.

The possible outcome of the study is the development of the physical model facilities that will be used for short and long term hydraulic research activities. A quantitative knowledge on scour and scour pattern along the revetment and a quantitative estimate of the initial and final positions of protection material is obtained. The guideline that is proposed for hydraulic design of a revetment structure like Hexagonal Armour Unit can be used as preliminary basis for river bank protection works in Bangladesh.

1.3 Brief Methodology

The physical model facilities have been developed for short term and long term research activities including morphological studies, bank protection and river

training works. This setup covers approximately 61 m length and 11.5 m width. It is equipped with storage pools for water supply, upstream and downstream reservoirs, sedimentation tank, pumps, recirculating canal, measuring devices like weirs, water level recorders, point gauges, side rails and carriages etc.

The study on bank protection work by revetments will be done on the above mentioned physical model setup for different velocity and bank slope condition. Cubical blocks and Hexagonal blocks will be used as revetments in the present modelling work. For the study purpose, measurement of discharge, velocity, water level and bed level will be taken at selected stations. Photographic recording of the erosion at the protection work will also be done.

1.4 Organization of the Study

The present study is organized as follows. In Chapter 1, the objectives, scope and brief methodology of the study are described. In Chapter 2, literature review on physical modelling, modelling procedure and details of bank protection work using revetment structures are presented. Next, in Chapter 3, descriptions of the salient features of the components of the physical model are presented. Preparation of the setup for study on bank protection works by revetment is presented in Chapter 4. In Chapter 5, analysis of experimental results from the study on bank protection works by revetments is presented. Chapter 6 provides the conclusions on the present study and recommendations for the future research work.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 *Introduction*

Physical modelling requires the knowledge of the phenomena involved to arrive at a useful model. Usually the dimensions of a physical model are smaller than the dimensions of the corresponding hydraulic phenomena in nature i.e. in prototype. While dealing with real problem, the principle of the use of physical models consists of the possibility to represent the real problem on a smaller scale in such a way that the phenomena in the scale model are similar in model and prototype.

2.2 *Reasons for Physical Modelling*

The most important reason of physical modelling of river problems is economy. Normally hydraulic structures such as bridges, culverts, barrages groynes, revetments etc. are constructed within the domain of rivers. These structures are constructed with specific goals and objectives. Most of the cases, these structures are very expensive. So, such an important and expensive structure does not permit an error in design or misinterpretation of actually prevailed condition in the field. The possibility of misinterpretation of design parameters can greatly be reduced or even eliminated with the help of physical modelling. It helps to avoid danger and financial losses which arise from the failure of structure due to improper design.

Sometimes physical modelling helps to reduce the cost of a structure. Due to uncertainty of design parameters, the designer keeps a large safety factor making the structure more expensive. But once the better understanding of the process and certain behavior of the design parameter are attained by physical modelling, the designer can change or reduce the size, spacing, orientation etc. of the structure with confidence and consequently the cost will be reduced. So properly done physical model investigation can act as a good tool for optimum design from structural, hydraulic and economic points of view of hydraulic structures constructed in the domain of a river.

2.2.1 Classification of model

Physical model is generally classified as-

- (a) Undistorted model
- (b) Distorted model
- (c) Fixed bed model
- (d) Movable bed model

(a) Undistorted model

When all the similarity requirements are fulfilled in a model or if a model is perfectly similar to its prototype then the model is said to be undistorted or true scale model. In an undistorted model all the vertical lengths (including diameter of the bed material) and horizontal lengths are reduced with a constant scale factor and the slope ratio will be equal to unity. Results obtained from this type of model are quantitative and can be used successfully in the prototype.

(b) Distorted model

A model is said to be distorted if one or more similarity requirements are not fulfilled. A distorted model is in fact a compromise between a number of requirements and limitations. The rational approach will lead into the vicinity of the optimum point. But trial and error is only the way to find the best compromise.

In a distorted model vertical lengths and horizontal lengths of prototype are reduced to model with different scale factors and slope ratio deviates from unity. Results obtained from a distorted model are deviated from its prototype to some extent and does not represent to its prototype directly and that is why the results obtained from this type of model are qualitative. So profound knowledge, extensive experiences and fair judgments are essential to transfer data of distorted model to its prototype for use. To get precise result an undistorted model is expected but due to some physical and practical problems such as existing model facilities, precise and accuracy in measurements, fulfillment of hydraulic conditions, availability and characteristics of bed material etc. distortion (geometrical distortion, material distortion and the distortion of configuration) is allowed in some cases.

(c) Fixed-bed model

The fixed-bed model is designed with Froude's model law to hold strict dynamic similarity under the framework of rough turbulent flow. In this type of model, the similarity of boundary resistance should also be ensured. The fixed-bed indicates that sediment movement and bed forming processes are out of concern in designing this type of model. During the model construction, fixed bed action can be achieved with any suitable mechanism with the assurance that the required roughness in the model is properly reproduced.

(d) Movable-bed model

The name movable-bed implies that in this type of model due attention should be paid to sediment transport, erosion, deposition and bed-forming processes. The design criteria of this type of model are almost similar to that of fixed-bed model with a little exception. The exception is that in the movable-bed model extensive attention should be paid to attain similarity in incipient motion and bed forming processes and thereby ensuring the bed-load movement in the model. To fulfill the similarity requirement regarding bed-load movement, it is normally found to use light-weight material such as coal- dust, saw-dust etc.

2.2.2 Scaling procedure of physical modelling

Scaling procedure in physical modelling should be in such a way that all the behavior and characteristics in the prototype could be reproduced in the model with a reduced scale. To achieve authentic results from the physical modelling, it is necessary to reproduce the geometric, kinematic and dynamic similarity in the model and prototype.

2.2.2.1 Determination of vertical length scale

(a) Froude's Model Law

Froude's number is defined as the square root of the ratio of inertia force to viscous force. Mathematically it can be expressed in the following way-

$$F = V/(gh)^{1/2} \quad \text{Eq. 2.1}$$

where, F = Froude's number

V = Average velocity of the cross section

g = Acceleration due to gravity

h = Depth of flow

From similarity requirements the Froude's number in the model and in the prototype must be the same i.e.

$$F_p = F_m$$

$$\text{or, } V_p/(g_p x h_p)^{1/2} = V_m/(g_m x h_m)^{1/2} \quad \text{Eq. 2.2}$$

where, p and m indicates prototype and model respectively. After rearrangements and considering $g_p = g_m$, from Equation 2.2, one can write-

$$\eta_v = (\eta_h)^{1/2} \quad \text{Eq. 2.3}$$

where $\eta_v = V_p/V_m$, velocity scale

$\eta_h = h_p/h_m$, vertical length scale

Equation 2.3 is the Froude's law for physical modelling in mathematical form. Almost all the physical modelling of open channel flow is normally done on the basis of Froude's model law.

(b) Reynold's model law

Reynold's law simply relates viscous force with inertia force through the following mathematical relation-

$$R = Vh/\nu \quad \text{Eq. 2.4}$$

where, R = Reynold's number

V = Average velocity in the cross section (m/s)

ν = Kinematic viscosity (m^2/sec)

h = Depth of flow (m)

According to the similarity requirements, Reynold's number in the model and in the prototype must be the same i.e.

$$R_p = R_m$$

$$\text{or } V_p h_p / \nu_p = V_m h_m / \nu_m \quad \text{Eq. 2.5}$$

After rearrangements, from equation 2.5, one can write-

$$\eta_v \eta_h \eta_v^{-1} = 1 \quad \text{Eq. 2.6}$$

where $\eta_v = \nu_p/\nu_m$, scale ratio for kinematic viscosity

Equation 2.6 is the Reynold's law for physical modelling.

It is important to note that simultaneous fulfillment of Froude's model law i.e. equation 2.3 and Reynold's model law i.e. equation 2.6 in the model is quite impossible. So an alternate way should be followed to avoid or to minimize the effect of viscous force in the model.

The effect of viscous force is no longer significant when the flow in the open channel is hydraulically rough and turbulent. So to avoid or to minimize the effect of viscous force, the flow in the model must be hydraulically rough and turbulent. Normally flow in open channel is turbulent when $R > 2000$ but for good minimization of viscous effect, Reynold's number should not be less than 5000 in the model.

So, in designing a physical model, one should strictly follow the Froude's model law to hold strict dynamic similarity with the assurance that the flow in the model is hydraulically rough and turbulent.

With the facts described above, the Reynold's model law needs not to be used in the scaling procedure of physical modelling and another useful relation combining Reynold's number can be form in the following way-

$$R_p/R_m = V_p h_p \nu_m / V_m h_m \nu_p \quad \text{Eq. 2.7}$$

After rearrangements equation 2.7 can be written as-

$$\eta_R = \eta_h^{3/2} \eta_v^{-1} \quad \text{Eq. 2.8}$$

where, $\eta_R = R_p/R_m$, Scale for Reynold's number

Equation 2.8 is an important relation for the selection of model scale.

From equation 2.8 it is clear that one can not make vertical scale of the model large enough beyond a critical value after which the flow in the model will be no longer turbulent; A slight increment in the vertical scale causes a rapid reduction of model Reynold's number which in turn qualifies viscous force to affect the flow condition in the model.

Surface tension may have a significant effect on the fluid flow depending on the size of the model and flow conditions. Under turbulent flow condition, the effect of surface tension is usually negligible. But if the model is very small, the effect of surface tension should also be investigated.

The effect of surface tension can be evaluated in term of Weber number, which is the ratio of inertia force to surface tension.

Mathematically-

$$W = \rho V^2 h / \sigma \quad \text{Eq. 2.9}$$

where W = Weber number

σ = Surface tension

From similarity requirements, equation 2.9 can be written as-

$$W_p = W_m$$

$$\text{or, } \rho_p V_p^2 h_p / \sigma_p = \rho_m V_m^2 h_m / \sigma_m \quad \text{Eq. 2.10}$$

After rearrangements equation 2.10 can be written as—

$$\eta_\rho \eta_v^2 \eta_h \eta_\sigma^{-1} = 1 \quad \text{Eq. 2.11}$$

where $\eta_\rho = \rho_p / \rho_m$, Fluid density scale

$\eta_\sigma = \sigma_p / \sigma_m$, Surface tension scale

To avoid the effect of surface tension, the value of Weber number should have sufficiently large value in the model. The scale of Weber number is given by-

$$\eta_w = \eta_h^2 \quad \text{Eq. 2.12}$$

This equation simply states that Weber number ratio decreases too rapidly in the model with the slight increment in the vertical scale i.e. with little increment in vertical scale Weber number in the model reduces too largely and hence surface tension tends to be more dominating.

2.2.2.2 Determination of friction and slope ratios

The average velocity in the cross section can be calculated from the following equation (Chezy's equation)-

$$V = (8ghS/f)^{1/2}$$

Eq. 2.13

where, V = Average velocity in the cross section

S = Average slope of the water surface

f = Darcy-Weisbach friction factor

The friction factor may be calculated from the following relations, suggested by Keulegan and White Cole Brook respectively-

$$f = (2.21 + 2.03 \log(h/K_s))^{-2} \quad \text{Eq. 2.14a}$$

$$f = (2 \log(14.8 h/K_s))^{-2} \quad \text{Eq. 2.14b}$$

Where, K_s stands for the height of the bed roughness.

Equations 2.14a and 2.14b give comparable results.

Here, the value of K_s can be taken as 2 to $2.5D_{50}$.

Where D_{50} is the grain diameter corresponding to 50 percents finer.

Here it can be mentioned that, the value of roughness projection, K_s , cited above is quite approximate and no generally accepted method for estimating correct value of K_s in open channel flow has yet been put forward. So accuracy of the selected value of K_s in open channel flow largely depends on the engineering judgment and profound experience in the field of open channel hydraulics. But since K_s contributes to the friction factor (f) through logarithmic function, small error in estimating the value of K_s does not cause substantial error.

From equation 2.13, for the model and the prototype one can write-

$$\eta_v^2 \eta_f = \eta_h \eta_s \quad \text{Eq. 2.15}$$

From equation 2.3 and 2.15 the following relation can be obtained-

$$\eta_s = \eta_f \quad \text{Eq. 2.16}$$

This relation simply states that the scale factor for the water surface slope and bed friction are equal.

2.2.2.3 Density of the model bed materials

From the similarity requirements, it is essential that the Shield's parameter in the model and in the prototype must have the same value. In connection of Shield's parameter the incipient motion of the bed materials can be defined in term of bed mobility. The bed mobility in the model and in the prototype should be the same at the corresponding depth to achieve similarity in incipient motion of the bed materials and bed load transport in the model and in the prototype.

The bed mobility is defined as-

$$M = \psi / \psi_c \quad \text{Eq. 2.17}$$

where, M = River bed mobility

ψ = Shield's parameter

The Shield's parameter can be defined as-

$$\psi = \gamma h S / ((\gamma_s - \gamma) D_{50}) \quad \text{Eq. 2.18}$$

where, γ = Specific weight of water

γ_s = Specific weight of bed materials

ψ_c = Critical Shield's parameter whose value can be obtained from Shield's diagram corresponding to the particle Reynold's number.

Particle Reynold's number can be calculated from the following relation—

$$R_* = V_* K_s / \nu \quad \text{Eq. 2.19}$$

Where, R_* = Particle Reynold's number

$V_* = (ghS)^{1/2}$, Shear velocity

Particle Reynold's number can be expressed in term of Reynold's number by the following equation-

$$R_* = R(f/8)^{1/2} (K_s/h) \quad \text{Eq. 2.20}$$

In the most of the cases, flow in the natural river is hydraulically rough. So from the similarity requirements the flow in the model should also be hydraulically rough.

Rough turbulent flow in open channel can be defined in the following two ways-

(1) The thickness of viscous sublayer can be defined by

$$\delta = 11.6 K_s / R_*$$

Eq. 2.21

If $\delta > K_s$, then the flow is said to be hydraulically smooth and in this condition the flow characteristics depends on the viscous force.

If $\delta < K_s$, then the flow is said to be hydraulically rough and in this condition the flow characteristics is independent of the viscous force and entirely depends on the roughness projection.

(2) Turbulent flow can be divided into three different types-

(a) Turbulent flow with smooth bed when $R_* < 5$

(b) Turbulent flow with transitional bed when $5 < R_* < RR$

Where, $RR = 60$, for sand grain type of roughness

$RR = 100$, for any other type of roughness

(c) Turbulent flow with rough bed when $R_* > RR$

In this context it can be mentioned that the flow conditions may be hydraulically smooth even when the surface is rough provided that the roughness projections are small enough to be deeply buried within the viscous sublayer. As the Reynolds number increases and this sublayer shrinks, the roughness projections become a significant factor and the flow enters a transitional stage, culminating when the projections break through the viscous sublayer and dominate the flow behavior, which is then termed as fully developed rough flow.

From similarity requirements and from equation 2.17 one can write

$$M_p = M_m \quad \text{or}$$

$$\psi_p / \psi_{cp} = \psi_m / \psi_{cm} \quad \text{or}$$

$$\psi_m = \psi_p \times \psi_{cm} / \psi_{cp}$$

Eq. 2.22

Where ψ_{cp} and ψ_{cm} are known and ψ_p can be calculated from equation 2.18 and hence from equation 2.22 ψ_m can be found out.

Finally the scale factor for the Shield's parameter can be carried out as-

$$\eta_\psi = \psi_p / \psi_m$$

Eq. 2.23

It is clear that this value of η_ψ should be equal to unity to fulfill the similarity requirements. But due to Reynold's number distortion, the value of η_ψ may be deviated from unity.

From equation 2.18 the following important relation can be formulated-

$$\eta_\psi = \eta_h \eta_s \eta_{\gamma_s - \gamma}^{-1} \eta_{D_{50}}^{-1} \quad \text{Eq. 2.24}$$

where, $\eta_{\gamma_s - \gamma}^{-1} = (\gamma_s - \gamma)_p / (\gamma_s - \gamma)_m$, scale factor for specific weight of bed materials.

In the equation 2.24; η_ψ , η_h , η_s and $\eta_{D_{50}}$ are known and hence $\eta_{\gamma_s - \gamma}$ can be computed and once the value of $\eta_{\gamma_s - \gamma}$ is computed, the specific weight of the model bed materials can be found out.

2.2.2.4 Determination of horizontal length scale

Normally horizontal length scale is selected on the basis of available space for the model and from this point of view the horizontal length scale is given by-

$$\eta_L = L_p / L_m \quad \text{Eq. 2.25}$$

If space for the model permits, the following relation can also be used to determine horizontal length scale-

$$\eta_s = \eta_h / \eta_L \quad \text{Eq. 2.26}$$

Equation 2.26 gives the value of S_L for the similarity requirements and equation 2.25 gives the value of S_L for available space limitation.

2.2.2.5 Discharge and Time scale

Scale ratio for area is given by-

$$\eta_A = \eta_L \eta_h \quad \text{Eq. 2.27}$$

Discharge ratio can be evaluated by-

$$\eta_W = \eta_A \eta_V \quad \text{Eq. 2.28}$$

From equation 2.27 and 2.28 one can write-

$$\eta_W = \eta_L \eta_h^{3/2} \quad \text{Eq. 2.29}$$

Equation 2.29 gives discharge ratio for distorted model.

For an undistorted model, $\eta_L = \eta_h$ and equation 2.29 can be written as-

$$\eta_W = \eta_h^{5/2} \quad \text{Eq. 2.30}$$

Reference time related to sedimentation process is defined by-

$$T_s = Lh / (Q_s / (\gamma_s - \gamma)) \quad \text{Eq. 2.31}$$

where, Q_s = Rate of bed load discharge and

$$Q_s = 8(\psi - \psi_c)^{3/2} (\gamma_s - \gamma)^{3/2} K_s^{3/2} (g / \gamma)^{1/2} \quad \text{Eq. 2.32}$$

From equation 2.31 and 2.32 one can write-

$$T_s = (Lh) / (8(\psi - \psi_c)^{3/2} (\gamma_s - \gamma)^{1/2} K_s^{3/2} (g / \gamma)^{1/2}) \quad \text{Eq. 2.33}$$

Converting this into time scale one can write-

$$\eta_{TW} = \eta_L \eta_h \eta_{\psi - \psi_c}^{-3/2} \eta_{\gamma_s - \gamma}^{-1/2} \eta_{K_s}^{-3/2} \quad \text{Eq. 2.34}$$

(assuming $\gamma_p = \gamma_m$ and $g_p = g_m$)

Equation 2.34 gives time scale related to sedimentation process. Reference time related to strict hydraulic aspects (also called reference time related to water) is defined by-

$$T_w = (Lh) / q \quad \text{Eq. 2.35}$$

where, q = discharge per unit width

$$= hV$$

So equation 2.35 can be written as-

$$T_w = L / V \quad \text{Eq. 2.36}$$

Converting this into time scale one can write-

$$\eta_{TW} = \eta_L / \eta_V \quad \text{Eq. 2.37}$$

Equation 2.37 gives time scale related to strict hydraulic aspects.

2.2.3 Points for selecting model scale

1. Froude's number in the model and in the prototype must be the same to hold strict dynamic similarity.
2. Flow in the model must be turbulent to avoid the effect of viscous force in the model.
3. The Weber number in the model should have sufficiently, large value to overcome the effect of surface tension. In the river model, this is not too much important because in the turbulent flow condition, the effect of surface tension is always negligible. But in the case of very small model, the effect of surface tension should be investigated.

4. As the most of flows in the prototype occur in hydraulic rough condition, flow in the model should also be in hydraulic rough condition.
5. Utmost efforts should be provided to avoid distortion of the model, but when conditions compel to have it, care should be taken to keep distortion at the minimum level.
6. Similarity in the incipient motion of bed material and bed load transport between the model and the prototype should be ensured. In case of mobile bed model, care should be taken to ensure bed load movement.
7. Depth of water in the model should be sufficient for convenient measurements of velocity, scour etc.
8. During selection of model bed material efforts should be provided to use available material.
9. Horizontal length scale should be selected on the basis of available space to accommodate the model.
10. Size of the model bed material should not be too small otherwise cohesion may develop.
11. Existing water pumping capacity should be considered.
12. Capacity of existing wave generator should also be considered.

Lastly, accuracy of the model test results is the first priority. So one should not be deviated from the similarity requirements to fulfill all the constraints regarding available model bed materials, space available for the model, existing water pumping capacity and capacity of existing wave generator.

2.2.4 Model calibration

Even after designing a model with great care, it remains absolutely essential for distorted as well as undistorted models to be verified against some field data. Through verification of the model, one can understand the degree of representativeness of the model to its prototype i.e. how much closely the model is representing to its prototype.

A model can possibly be verified in the following ways

1. Verification of stage-discharge relationship at least at two points.

2. The discharge at which incipient motion occurs should be verified.
3. Verification of some local scour aspects known to occur in the prototype under some flow conditions if verification of sediment load is not possible.
4. Verify flow pattern in the model without any hydraulic structure in the light of observed flow pattern in the prototype in the same condition as that in the model. If necessary, some adjustments in the model roughness need to be made to simulate the flow pattern.
5. Verify flow velocity in the model at some particular points (without hydraulic structure) against authentically measured prototype velocity at the same points.

If model verification disproves similarity with prototype, a trial and error approach will be required to improve the model by adjusting the model roughness or by adjusting any adjustable parameter.

2.3 Bank Erosion and Bank Protection Works

The rivers of Bangladesh are quite unstable and continually erode one bank or build the other. By this process of erosion they have engulfed large areas of cities and towns, agricultural lands and embankments. The large discharge and heavy sediment load make these alluvial rivers unstable and constantly migrating laterally. It is estimated that about 1200 km of bank length of rivers of Bangladesh are subjected to erosion and of which about 565 km face severe erosion problem (BWDB, 1984). The summary of river bank erosion in Bangladesh are presented in Table 2.1.

Table 2.1 Summary of river bank erosion in Bangladesh

Name of River	No. of Location of bank erosion	Length of Erosion (km)
Brahmaputra-Jamuna	41	162.60
Ganges-Padma	26	94.50
Meghna	8	72.00
Teesta	11	34.90
Minor rivers	112	92.30
Flashy rivers	75	23.00
Tidal rivers	32	85.80
Cities	85	565.10

Source: Guide to planning and design of river training and bank protection works, BWDB (1993)

Table 2.2 presents different types of River Training Works used in Bangladesh to protect important towns from river bank erosion.

Table 2.2 River training works used in Bangladesh

Sl. No.	Name of Scheme	Type of protective Works
1.	Rajshahi Town Protection	a. Groyne, b. Spur, c. Brick Matressing
2.	Serajgonj Town Protection	a. Bank Revetment b. Groyne
3.	Khulna Town Protection	a. Bank Revetment
4.	Kurigram Town Protection	a. Groyne b. Bank Revetment c. Diversion Channel d. Closure Dam
5.	Kumarkhali Town Protection	a. Groyne
6.	Kustia Town Protection	a. Groyne
7.	Chandpur Town Protection	a. Bank Revetment b. Groyne
8.	Moulvibazar Town Protection	a. Bank Revetment b. Flood wall

Source: Guide to planning and design of river training and bank protection works, BWDB (1993)

2.4 Causes of Bank Recession

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

- (a) Washing away of soil particles of the bank by current or waves; this is called erosion.
- (b) Sliding due to the increase of the slope of bank as a result of erosion and scour.
- (c) Undermining of the toe of lower bank by currents, waves, swirls or eddies, followed by collapse of overhanging materials deprived of support; this is called scour.
- (d) Sloughing or sliding of slop when saturated with water; this is usually the case during floods of long duration.
- (e) Sliding due to seepage of water flowing back into the river after receding of flood; the internal shearing strength is considerably decreased owing to saturation and the stability is further decreased by the pressure of seepage flow.
- (f) Piping in a sub-layer due to movement of ground water to the river which carries away sufficient material with it.

2.5 Purpose and Objective of River Training Works

The purpose of river training is to stabilize the channel along a certain alignment with certain cross section for one or more of the following objectives:

- ◆ Safe and expeditious passage of flood flow
- ◆ Efficient transportation of suspended and bed load
- ◆ Stable river course with minimum bank erosion
- ◆ Sufficient depth and good course for navigation
- ◆ Direction of flow through a certain defined stretch of the river.

2.6 Types of River Training Works

Type of works to be done for the purpose of training a river reach is dependent on the objective and engineering principle chosen to be adopted in the river training programme. The usually adopted types of river training works are:

- Bank Revetment
- Groyne
- Guide Bund
- Artificial Loop Cut
- Island
- Retards
- Porcupine
- Bandals
- Biological Treatment

In the present study only the bank protection by revetment is considered and a detail description of this types of structures are given below:

2.7 Riprap

Riprap is defined as a protective cover consisting of a well graded mixture of rock, concrete blocks, or other material, usually dumped or hand-placed, applied on the slope and toe of the river bank so that the bank soil is protected from the actions of erosive forces of flowing water and dynamic actions of waves.

The types of slope protection or revetment include (Brown and Clyde, 1989):

- Rock riprap

- ❑ Rubble riprap
- ❑ Wire-enclosed rock (Gabions)
- ❑ Pre-formed blocks
- ❑ Grouted rock
- ❑ Paved Lining

2.7.1 Rock Riprap

Rock riprap is the most widely used type of revetment in the world. It is compatible with most environmental settings. The term "riprap" is most often used to refer to rock riprap. Rock riprap can be further subdivided by placement method into dumped riprap, hand-placed riprap, and plated riprap.

Dumped riprap is graded stone dumped on a prepared slope in such a manner that segregation will not take place. Dumped riprap forms a layer of loose stone; individual stones can independently adjust to shifts in or movement of the base material. The placement of dumped riprap should be done by mechanized means, such as crane and skip, dragline, or some form of bucket. End dumping from trucks down the riprap slope causes segregation of the rock by size, reducing its stability, and therefore, should not be used as a means of placement. The effectiveness of dumped riprap has been well established where it is properly installed, of adequate size, and suitable size gradation. Advantages associated with the use of dumped rock riprap include:

- The riprap blanket is flexible and is not impaired or weakened by minor movement of the bank caused by settlement or other minor adjustments.
- Local damage or loss can be repaired by placement of more rock.
- Construction is not complicated.
- When exposed to fresh water, vegetation will often grow through the rocks, adding esthetic and structural value to the bank material and restoring natural roughness.
- Riprap is recoverable and may be stockpiled for future use.

One drawback to the use of rock riprap revetments is that they are more sensitive than some other bank-protection schemes to local economic factors. For example,

freight/haul costs can significantly affect the cost of these revetments. Figure 2.1 illustrates a dumped riprap installation.



Figure 2.1 Dumped Rock Riprap (Source: Brown and Clyde, 1989)

Hand-placed riprap is stone laid carefully by hand or by derrick following a definite pattern, with the voids between the larger stones filled with smaller stones and the surface kept relatively even. The need for interlocking stone in a hand-placed revetment requires that the stone be relatively uniform in size and shape (square or rectangular).

Advantages associated with the use of hand-placed riprap include:

- The even interlocking surface produces a neat appearance and reduces flow turbulence at the water revetment interface.
- The support provided by the interlocking of individual stones permits the use of hand-placed riprap revetments on steeper bank slopes than is possible with the same size loose stone riprap.
- With hand-placed riprap, the blanket thickness can usually be reduced to 150 to 300 mm less than a loose riprap blanket, resulting in the use of less stone.

Disadvantages associated with hand-placed riprap include:

- Installation is very labor-intensive, resulting in high costs.
- The interlocking of individual rocks in hand-placed revetments results in a less flexible revetment; a small shift in the base material of the bank can cause failure of large segments of the revetment.

By their nature, hand-placed rock riprap revetments are more expensive to repair than are loose rock revetments.



Figure 2.2 Hand-placed Riprap (Source: Brown and Clyde, 1989)

Plated or keyed riprap is similar to hand-placed riprap in appearance and behavior, but different in placement method. Plated riprap is placed on the bank with a skip and then tamped into place using a steel plate, thus forming a regular, well organized surface. Experience indicates that during the plating operation, the larger stones are fractured, producing smaller rock sizes to fill voids in the riprap blanket.

Advantages and disadvantages associated with the use of plated riprap are similar to those listed above for hand-placed riprap. As with hand-placed riprap, riprap plating permits the use of steeper bank angles, and a reduction in riprap layer thickness (usually 150-300 mm less than loose riprap). Experience also indicates that riprap plating also permits the use of smaller stone sizes when compared with loose riprap. Like hand-placed riprap, riprap plating results in a more rigid riprap lining than loose riprap. This makes it susceptible to failure as a result of minor bank settlement. However, plated riprap installation is not as labor-intensive as that of hand-placed riprap.

Figure 2.3 illustrates a plated riprap installation under construction.



Figure 2.3 Plated or Keyed Riprap (Source: Brown and Clyde, 1989)

2.7.2 Rubble Riprap

Types of rubble that have been used as riprap include rock spoils, broken concrete, and steel furnace slag.

Rock spoils are often available from road cut or other excavation sites.

Broken concrete is available in areas undergoing widespread urban renewal involving the demolition of buildings and other structures made from concrete.

Steel furnace slag is sometimes available in the vicinity of steel smelting plants. Because it is usually considered to be a waste material, rubble is a very economical riprap material. Advantages and disadvantages to the use of rubble are quite similar to those listed previously for rock riprap.

The successful use of rubble as riprap requires good control on material quality. The quality of a rubble material includes its shape, specific weight, gradation, and durability (resistance to weathering). The shape of rubble riprap is often a problem (particularly concrete rubble). The length to width ratio of any riprap material should be 1:3 or less. Plating of rubble riprap will often break the material sufficiently to reduce the length to width ratio of most of the material to less than 1:3. The material's specific weight can be accounted for in the design procedure for sizing the material. However, in many instances the rubble material will not contain an

appropriate mix of particle sizes to form an adequate riprap material. This can be overcome by a crushing operation, or by plating the rubble after placement. The recommended placement method for rubble riprap is plating. The lack of adequate material durability can cause the failure of rubble riprap. Rock spoils consisting of a high percentage of shale or other materials consisting of weakly layered structures are not suitable. Also, materials subject to chemical breakdown or high rates of weathering are not suitable. Figure 2.4 illustrates a site where broken concrete was used as the riprap material.



Figure 2.4: Broken Concrete Riprap (Source: Brown and Clyde, 1989)

2.7.3 Wire-Enclosed Rock

Wire-enclosed rock, or gabion, revetments consist of rectangular wire mesh baskets filled with rock. These revetments are formed by filling pre-assembled wire baskets with rock, and anchoring to the channel bottom or bank. Wire-enclosed rock revetments are generally of two types distinguished by shape:

1. **Rock and Wire-Mattresses**, In mattress designs, the individual wire mesh units are laid end to end and side to side to form a mattress layer on the channel bed or bank. The gabion baskets comprising the mattress generally have a depth dimension which is much smaller than their width or length.

2. Block Gabions, On the other hand, are more equidimensional, having depths that are approximately the same as their widths, and of the same order of magnitude as their lengths. They are typically rectangular or trapezoidal in shape. Block gabion revetments are formed by stacking the individual gabion blocks in a stepped fashion.

As revetments, wire-enclosed rock has limited flexibility. They will flex with bank surface subsidence; however, if excessive subsidence occurs, the baskets will span the void until the stresses in rock-filled baskets exceed the tensile strength of the wire strands. At this point the baskets will fail.

The conditions under which wire-enclosed rock is applicable are similar to those of other revetments. However, their economic use is limited to locations where the only rock available economically is too small for use as rock riprap slope protection.

The primary advantages of wire-enclosed rock revetments include:

- ◆ Their ability to span minor pockets of bank subsidence without failure.
- ◆ The ability to use smaller, lower quality, and less dense, rock in the baskets.
- ◆ The primary disadvantages of wire-enclosed rock revetments include:
 - ◆ Susceptibility of the wire baskets to corrosion and abrasion damage.
 - ◆ High labor costs associated with fabricating and filling the wire baskets.
 - ◆ More difficult and expensive repair than standard rock protection.
 - ◆ Less flexibility than standard rock protection.

Besides its use as a general bank revetment, wire-enclosed rock in the form of either mattresses or blocks is also used as bank toe protection. In some instances the wire-enclosed rock is used alone for protection of the bank also. In other cases, the wire-enclosed rock is used as toe protection along with some other bank revetment.

The most common failure mechanism of wire basket revetments has been observed to be failure of the wire baskets. Failure from abrasion and corrosion of the wire strands has even been found to be a common problem when the wire is coated with plastic. The plastic coating is often stripped away by abrasion from sand, gravel, cobbles, or other sediments carried in natural stream flows (particularly at and near

flood stages). Once the wire has been broken, the rock in the baskets is usually washed away. To avoid the problem of abrasion and corrosion of the wire baskets, it is recommended that wire-enclosed rock revetments not be used on lower portions of the channel bank in environments subject to significant abrasion or corrosion.

An additional failure mechanism has been observed when the wire basket units are used in high-velocity, steep-slope environments. Under these conditions, the rock within individual baskets shifts downstream, deforming the baskets as the material moves. The movement of material within individual baskets will sometimes result in exposure of filter or base material. Subsequent erosion of the exposed base material can cause failure of the revetment system.

A common misconception with rock and gabion revetments is that a heavy growth of vegetation will occur through the stone and wire mesh. Experience indicates that in many cases there is not sufficient soil retained within the baskets to promote significant vegetative growth. The exception to this is in areas subjected to significant deposition of fine materials (such as in the vicinity of bars). In areas where the baskets are frequently submerged by an active flow, vegetative growth will not be promoted.

Wire-enclosed rock revetments are classified by geometry as mattress or block type revetments. Rock and wire mattress revetments consist of flat wire baskets. The individual mattress sections are laid end to end and side to side on a prepared channel bed or bank to form a continuous mattress. The individual basket units are attached to each other and anchored to the base material. Figure 2.5 illustrates a typical rock and wire mattress installation. Block gabion revetments consist of rectangular wire baskets which are stacked in a stepped-back fashion to form the revetment surface. Gabion baskets are best used as bank protection where the bank is too steep for conventional rock riprap revetments. Gabion baskets can be stacked to form almost

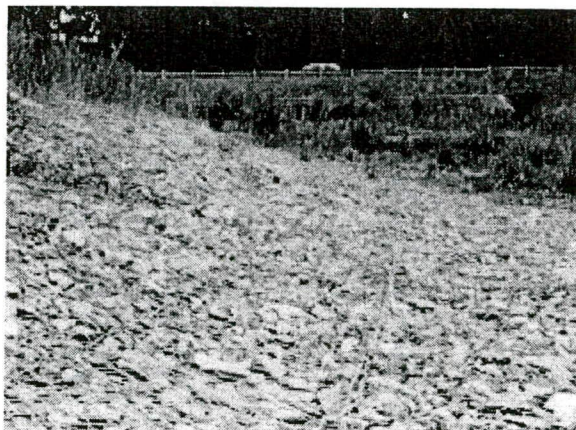


Figure 2.5 Rock and Wire Mattress Revetment (Source: Brown and Clyde, 1989)

vertical banks (looking much like retaining walls) making them useful in areas where the banks cannot economically be graded to the stable slope required for other riprap types. Figure 2.6 illustrates a typical block gabion installation.

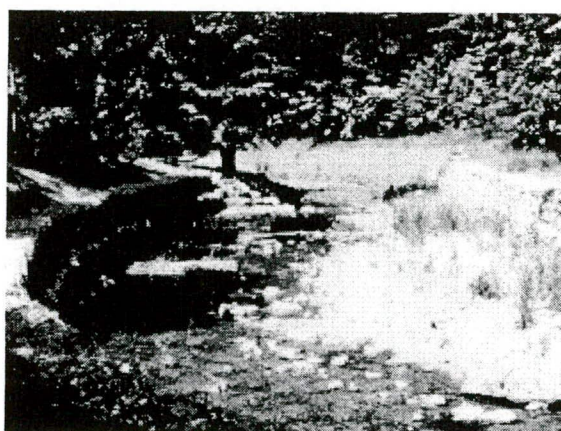


Figure 2.6 Gabion Basket Revetment (Source: Brown and Clyde, 1989)

2.7.4 Pre-Cast Concrete Block

Pre-cast concrete block revetments are a recent development. The pre-formed sections which comprise the revetment systems are butted together or joined in some fashion; as such, they form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation, but share certain common features. These features include flexibility, rapid installation, and provisions for establishment of vegetation within the revetment. The permeable nature of these revetments permits free draining of the bank materials; the flexibility, although limited, allows the mattress to conform to minor changes in the bank geometry. Their

limited flexibility, however, makes them subject to undermining in environments characterized by large fluctuations in the surface elevation of the channel bed and/or bank. Unlike wire-enclosed rock, the open nature of the pre-cast concrete blocks does promote volunteering of vegetation within the revetment.

The most significant drawbacks to the use of pre-cast concrete blocks are their limited flexibility and cost. As discussed above, their limited flexibility makes them subject to undermining in environments characterized by dynamic bed level fluctuations; failures have been observed where a corner or edge of the mattress is undercut, resulting in complete failure of the revetment. Pre-cast concrete block designs have also been shown to be expensive. For this reason, their use is usually limited to large rivers, areas where structures of significant value need to be protected, or where riprap is not readily available. Figure 2.7 illustrates a revetment consisting of pre-cast, interlocking blocks.



Figure 2.7 Pre-cast Concrete Block Mat (Source: Brown and Clyde, 1989)

2.7.5 Grouted Rock

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. Grouted rock is a rigid revetment; it will not conform to changes in the bank geometry due to settlement. As with other monolithic revetments, grouted rock is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted rock is not extremely strong; therefore, the loss of even a small area of bank support can cause failure of large portions of the revetment.

The use of grouted rock is usually confined to areas where rock of sufficient size for ordinary rock-slope protection is not economically available, or where a reasonably smooth revetment surface is desired (for reasons of safety or flow efficiency). The use of grouted rock can reduce the quantity of rock required; grouting anchors the rock, and integrates a greater material mass to resist the hydraulic forces it is exposed to. Also, if the embankment material is fine grained, grouting will eliminate the need for filter material that may be necessary with other rock slope-protection.

Grouting can double the cost per unit volume of stone. However, the ability to use smaller stones and thinner stone layers in grouted rock revetments than in ungrouted rock riprap offsets some of the additional cost of the grout. Figure 2.8 illustrates a grouted riprap installation.



Figure 2.8 Grouted Rock (Source: Brown and Clyde, 1989)

2.7.6 Paved Lining

Concrete pavement revetments are cast in place on a prepared slope to provide the necessary bank protection. Like grouted rock, concrete pavement is a rigid revetment which does not conform to changes in bank geometry due to a removal of foundation support by subsidence, undermining, outward displacement by hydrostatic pressure, slide action, or erosion of the supporting embankment at its ends. The loss of even small sections of the supporting embankment can cause complete failure of the revetment system. Concrete pavement revetments are also among the most expensive streambank protection designs. In the past, concrete pavement has been best utilized

as a subaqueous revetment (on the bank below the water surface) with vegetation or some other less expensive upper-bank treatment.

Concrete pavement revetments are required in some instances. The implied structural integrity of the concrete pavements makes them resistant to damage from debris, ice, and other floating objects. Their smooth surface also makes them useful in situations where hydraulic efficiency is of prime importance. They can also be erected on steep bank angles, making them useful in situations where bank grading is not practical. When installed properly, concrete pavement can provide a long useful life, requiring only a minimum of maintenance. Figure 2.9 illustrates a typical concrete slab revetment installation.

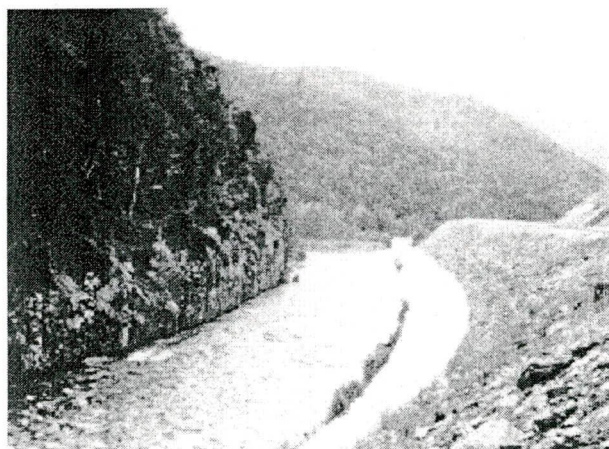


Figure 2.9 Concrete Pavement Revetment (Source: Brown and Clyde, 1989)

2.8 Erosion Mechanisms and Riprap Failure Modes

Prior to designing a bank stabilization scheme, it is important to be aware of common erosion mechanisms and riprap failure modes, and the causes or driving forces behind bank erosion processes. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Many causes of bank erosion and riprap failure have been identified. Some of the more common include abrasion, debris flows, water flow, eddy action, flow acceleration, unsteady flow, freeze/thaw, human actions on the bank, ice, precipitation, waves, toe erosion, and subsurface flows. However, it is most often a combination of mechanisms, which cause bank and riprap failure, and the actual

mechanism or cause is usually difficult to determine. Riprap failures are better classified by failure mode. Blodgett (1986) has identified classic riprap failure modes as follows:

- Particle erosion.
- Translational slide.
- Modified slump.
- Slump.

Particle erosion is the most common erosion mechanism. Particle erosion results when the tractive force exerted by the flowing water exceeds the bank materials ability to resist movement. In addition, if displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. This mound has been observed to cause flow concentration along the bank, resulting in further bank erosion. Particle erosion can be initiated by abrasion, impingement of flowing water, eddy action/reverse flow, local flow acceleration, freeze/thaw action, ice, or toe erosion. Probable causes of particle erosion include:

- ❑ Stone size not large enough.
- ❑ Individual stones removed by impact or abrasion.
- ❑ Side slope of the bank so steep that the angle of repose of the riprap material is easily exceeded.
- ❑ Gradation of riprap too uniform.

Figure 2.10 illustrates riprap failure by particle erosion.

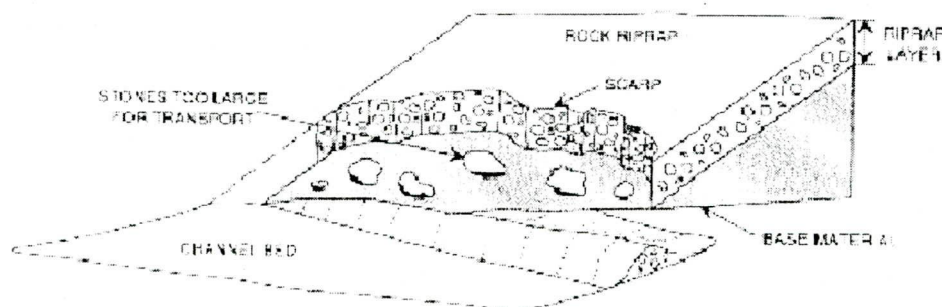


Figure 2.10 Particle Erosion Failure (Source: Brown and Clyde, 1989)

A **translational slide** is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane. The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. As the slide progresses, the lower part of riprap separates from upper part, and moves downslope as a homogeneous body. A resulting bulge may appear at the base of the bank if the channel bed is not scoured.

Translational slides are usually initiated when the channel bed scours and undermines the toe of the riprap blanket. This could be caused by particle erosion of the toe material, or some other mechanism, which causes displacement of toe material. Any other mechanism which would cause the shear resistance along the interface between the riprap blanket and base material to be reduced to less than the gravitational force could also cause a translational slide. It has been suggested that the presence of a filter blanket may provide a potential failure plane for translational slides (Blodgett, 1986). Probable causes of translational slides are as follows:

- Bank side slope too steep.
- Presence of excess hydrostatic (pore) pressure.
- Loss of foundation support at the toe of the riprap blanket caused by erosion of the lower part of the riprap blanket (Blodgett, 1986).

Figure 2.11 illustrates a typical translational slide.

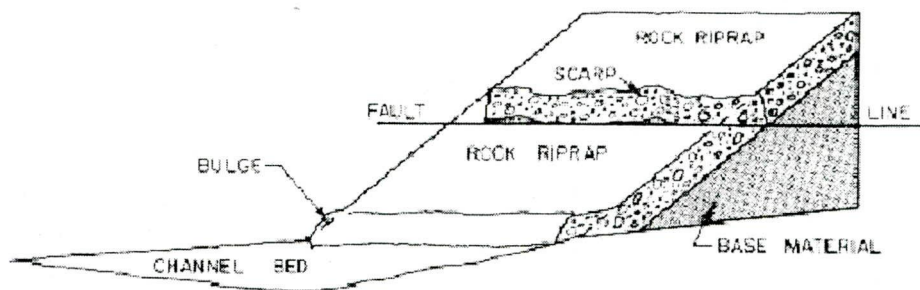


Figure 2.11 Translational Slide Failure (Source: Brown and Clyde, 1989)

The failure of riprap referred to as modified slump is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to

the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion. Probable causes of modified slump are:

- Bank side slope is so steep that the riprap is resting very near the angle of repose, and any imbalance or movement of individual stones creates a situation of instability for other stones in the blanket.
- Material critical to the support of upslope riprap is dislodged by settlement of the submerged riprap, impact, abrasion, particle erosion, or some other cause (Blodgett, 1986).

Figure 2.12 illustrates a modified slump failure.

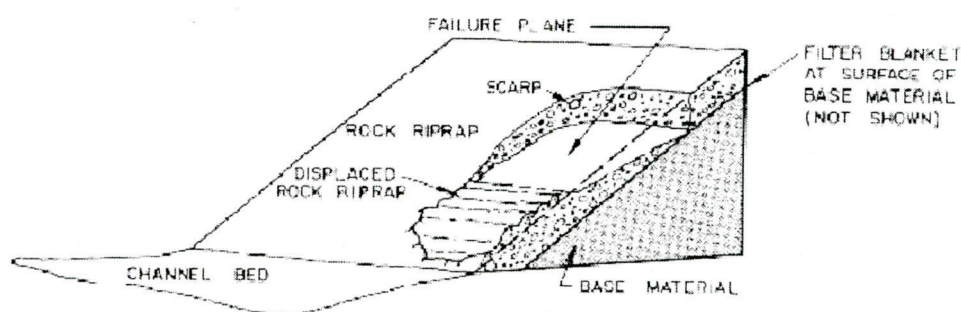


Figure 2.12 Modified Slump Failure (Source: Brown and Clyde, 1989)

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures (Figure 2.13) is related to shear failure of the underlying base material that supports the riprap revetment. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material. Probable causes of slump failures are: Non-homogeneous base material with layers of impermeable material that act as a fault line when subject to excess pore pressure. Side slope too steep, and gravitational forces exceed the inertia forces of the riprap and base material along a friction plane (Blodgett, 1986).

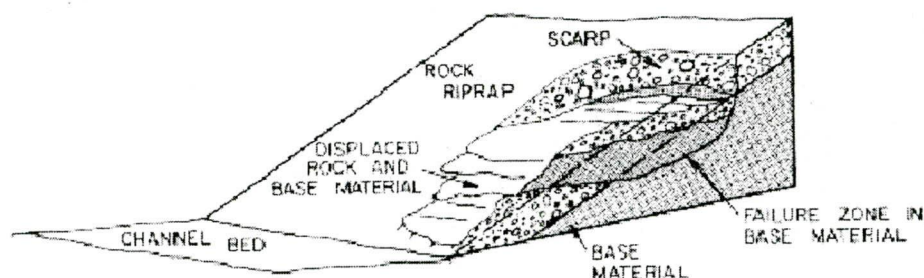


Figure 2.13 Slump Failure (Source: Brown and Clyde, 1989)

2.9 Extent of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

2.9.1 Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criteria, it demands serious consideration. Review of existing bank protection sites has revealed that a common misconception in streambank protection is to provide protection too far upstream and not far enough downstream.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection scheme be extended a minimum of one channel width beyond the observed erosion limits.

In curved channel reaches, the scars on the channel bank can be used to establish the upstream limit of erosion. Here again, a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to

define. Since the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

The extent of bank protection can also be influenced by existing channel controls. The most common situation encountered is the existence of a bridge somewhere along the bend. If the bridge has an abutment immediately adjacent to the channel bank, it will act as a control point with respect to channel stability. The location of the bridge abutment (or other channel control such as a rock outcrop) will usually define the downstream limit of active channel movement. If the control point does not cause significant flow contraction, or there is no significant flow expansion downstream of the control, the bank revetment should be terminated approximately one channel width downstream of the control. However, if significant flow contraction and/or expansion is occurring in the vicinity of the control, the protection should be continued downstream for a distance equal to four times the constricted channel width at the control.

2.9.2 Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

Design Height

The design height of a riprap installation should be equal to the design highwater elevation plus some allowance for freeboard. Freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccounted factors. Some such factors include:

- ☐ Wave action (from wind or boat traffic).
- ☐ Superelevation in channel bends.
- ☐ Hydraulic jumps.
- ☐ Flow irregularities due to piers, transitions, and flow junctions.

In addition, erratic phenomena such as unforeseen embankment settlement, the accumulation of silt, trash, and debris in the channel, aquatic or other growth in the channels, and ice flows should be considered when setting freeboard heights. Also, wave run-up on the bank must be considered.

There are many factors which must be considered in the selection of an appropriate freeboard height. As a minimum, it is recommended that a freeboard elevation of .30 to .61 m be used in unconstricted reaches, and .61 to .91 m in constricted reaches

Toe Depth

The undermining of revetment toe protection has been identified as one of the primary mechanisms of riprap revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation as well as natural scour and fill processes.

2.10 Riprap Size

2.10.1 Rock Size

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter. Relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces are described below.

Particle Erosion

Particle erosion is a hydraulic phenomenon which results when the tractive force exerted by the flowing water exceeds the riprap materials ability to resist motion. It is this process that the riprap design relationships presented in this section were developed for.

Two methods or approaches have been used historically to evaluate a materials resistance to particle erosion. These methods are:

1. Permissible velocity approach:

Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity.

2. Permissible tractive force (shear stress) approach.

The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary.

By Chow's definition, permissible tractive force is the maximum unit tractive force that will not cause serious erosion of channel bed material from a level channel bed. Permissible tractive force methods are generally considered to be more academically correct; however, critical velocity approaches are more readily embraced by the engineering community.

2.10.2 Dimension of Cubical Block

There are number equations available to determine the size of the revetment material to the velocity impinging on it. Some of the most common equations are as follows:

1. Isbash

$$W = \frac{4.1 \times 10^{-5} S_s V^5}{(S_s - 1)^3 \cos^3 \theta} \quad (\text{FPS unit})$$

2. California State Highways (CHS)

$$W = \frac{2 \times 10^{-5} S_s V^6}{(S_s - 1)^3 \sin^3 (70 - \theta)} \quad (\text{FPS unit})$$

3. Neill

$$D = 0.034V^2 \quad (\text{metric unit})$$

4. PIANC

$$D = \frac{0.7V^2}{g(S_s - 1)\cos\theta\left(1 - \frac{\tan^2\theta}{\tan^2\phi}\right)^{0.5}} \quad (\text{FPS or metric unit})$$

5. JMBA

$$D = \frac{0.7V^2}{g(S_s - 1) \left[\log\left(\frac{6h}{D}\right) \right]^2 \left[1 - \left(\frac{\sin \theta}{\sin \phi} \right)^2 \right]^{0.5}} \quad (\text{FPS or metric unit})$$

where, in these equations,

W = Weight of individual stone (lb. wt.)

D = Diameter of stone (ft or m)

D_n = Dimension of cube (ft or m)

V = Velocity (ft/s or m/s)

h = Depth of water (ft or m)

S_s = Specific gravity of stone

θ = Slope of bank

φ = Angle of repose of revetment materials

Ψ = Shields constant

g = Gravitational acceleration (ft/s² or m/s²)

6. Pilarczyk

The general formula for the design against current loads is given by,

$$D_n \geq \frac{0.035 \bar{u}^2}{\Delta_m \cdot 2g} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{K_S \cdot \Psi_{cr}} \quad (\text{metric unit})$$

where,

D_n (m) = characteristic size of the revetment cover layer material (single unit size for loose elements, thickness of mattress system)

$\bar{u}$ (m/s) = depth averaged flow velocity

Δ_m = relative density of submerged material = (ρ_s - ρ_w) / ρ_w

g (m/s²) = acceleration due to gravity

φ_{SC} = stability factor for current

Ψ_{cr} = critical shear stress parameter

K_t = turbulence factor

K_h = depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio:

$$K_h = 2.[\log(\frac{12h}{k_r})]^{-2} \text{ with } k_r = D_n \text{ for relatively smooth material}$$

$$K_s = \text{bank normal slope factor } K_s = K_s = \sqrt{1 - (\frac{\sin \alpha}{\sin \varepsilon_s})^2}$$

α ($^\circ$) = slope angle of bank or structure

ε_s (o) = angle of repose considering the material specific internal friction

2.10.3 Hexagonal Blocks

The Hexagonal Armour Unit, developed from a reappraisal of conventional theories of armour layer behaviour (Brown, 1978), was introduced at the coastal structure conference (Brown, 1979). These units enable many more cost-saving decisions to be made compared to contemporary units, and can achieve economies in every stage of construction (Brown, 1983). As the failure modes of these units depend on the aspect ratio (width to depth ratio) of the units and unit porosity, therefore, these parameters along with the others such as mass and densities of the units should also be taken into consideration. The thickness of the revetment can be calculated by using the following equation for Hexagonal armour unit under wave attack by replacing the design wave height by,

$$t = H/(1-p)C_B\phi(a)(s-1)$$

where, t = Armour layer thickness

H = Design wave height

p = armour layer porosity

C_B = Hydraulic performance coefficient of the layer

$\phi(a)$ = an angle function approx. $= \cot \alpha^{1/3}$

s = relative density of the armour unit

2.11 Filter Design

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils. For areas above the water line, filters also prevent surface water from

causing erosion (gullies) beneath the riprap. A filter should be used whenever the riprap is placed on noncohesive material subject to significant subsurface drainage (such as in areas where water surface levels fluctuate frequently and in areas of high groundwater levels).

The proper design of granular and fabric filters is critical to the stability of riprap installations on channel banks. If openings in the filter are too large, excessive flow piping through the filter can cause erosion and failure of the bank material below the filter. On the other hand, if the openings in the filter are too small, the build-up of hydrostatic pressures behind the filter can cause a slip plane to form along the filter resulting in massive translational slide failure.

2.11.1 Granular Filters

The requirement of a granular filter to keep the subsoil particles from invading the filter significantly are based on particle size. The filter criterion relates the grading of the filter to that of the subsoil. Therefore granular filters are to be designed in successively coarse layers proceeding outward from the underlying finest soil. The first layer should hold the subsoil, while each following layer has to be able to hold the underlying one. The outer layer must, however, also be stable under the prevailing hydraulic loads.

Some of the developed filter rules are present below:

BWDB:

$$d_{15} \leq 1.35 \text{ mm}$$

$$d_{50} < (1.8 \text{ mm} - 5.4 \text{ mm})$$

$$d_{15} \leq (0.54 \text{ mm} - 1.8 \text{ mm})$$

US Bureau of Reclamation:

$$d_{50} < 0.9 \text{ mm} - 1.8 \text{ mm}$$

$$U < 5$$

US Corp of Engineers:

$$d_{15} < 1.35 \text{ mm}$$

$$d_{15} < 1.8 \text{ mm}$$

$$d_{50} < 4.5 \text{ mm}$$

Pilarezyk:

$d_{15} < (1.08 \text{ mm} - 1.35 \text{ mm})$ (stability criterion)

$d_{15} > (0.36 \text{ mm} - 0.45 \text{ mm})$ (permeability criterion)

$d_{50} < (3.6 \text{ mm} - 4.5 \text{ mm})$

$U < 10$ (for no migration)

2.11.2 Fabric Filters

Synthetic fabric filters have found considerable use as alternatives to granular filters.

Advantages and disadvantages of synthetic fabric filters are discussed below.

Advantages relevant to the use of fabric filters have been identified:

- Installation is generally quick and labor-efficient.
- Fabric filters are more economical than granular filters.
- Fabric filters have consistent and more reliable material quality.
- Fabric filters have good inherent tensile strength.
- Local availability of suitable granular filter material is no longer a design consideration when using fabric filters.

Disadvantages include:

- Filter fabrics can be difficult to lay under-water.
- Installation of some fabrics must be undertaken with care to prevent undue ultraviolet light exposure.
- The life of the fabric in a soil environment is as yet unproven over the lifetime of a normal engineering project.
- Bacterial activity within the soil or upon the filter can control the hydraulic responses of a fabric filter system.
- Experimental evidence indicates that when channel banks are subjected to wave action, non-cohesive bank material has a tendency to migrate downslope beneath fabric filters; this tendency was not observed with granular filters.
- Fabric filters may induce translational or modified slump failures when used under rock riprap installed on steep slopes.

Fabric filters have a definite advantage over granular filters in many applications.

The primary justification is economic; the U.S. Army Corps of Engineers has found

geotextiles to be more cost effective than granular filters in many instances. This is particularly true in areas where a good source of gravel is not convenient

2.12 Edge Treatment

The edges of riprap revetments (head, toe, and flanks) require special treatment to prevent undermining.

Flanks: The flanks of the revetment should be designed as illustrated in Figure 2.14. The upstream flank is illustrated in part (b) and the downstream flank in part (c) of Figure 2.14. An alternative to the upstream flank section illustrated in part (b) is to fill the compacted fill area with riprap.

Toe: Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Figure 2.15. The toe material should be placed in a toe trench along the entire length of the riprap blanket as illustrated in Figure 2.15. Where a toe trench cannot be dug, the riprap blanket should terminate in a thick, narrow stone toe at the level of the streambed (see alternate design in Figure 2.15). Care must be taken during the placement of the stone to ensure that the toe material does not mound and form a low dike; a low dike along the toe could result in flow concentration along the revetment face which could stress the revetment to failure. In addition, care must be exercised to ensure that the channel's design capability is not impaired by placement of too much riprap in a toe mound. The size of the toe trench or the alternate stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will) the stone in the toe will launch into the eroded area. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 1V:2H. The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 1V:2H) to the anticipated depth of scour.

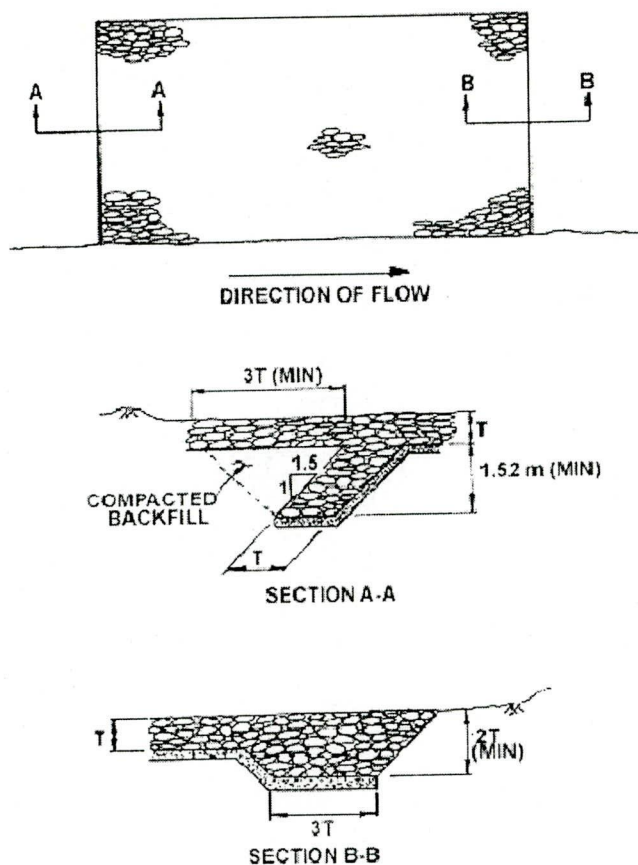


Figure 2.14 Typical Riptap Installation (Plan and Flank Detail) (Source: Brown and Clyde, 1989)

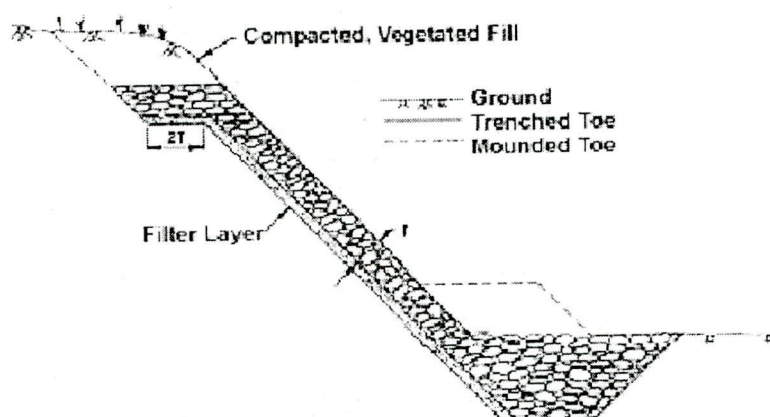


Figure 2.15 Typical Riptap Installation (bank protection only) (Source: Brown and Clyde, 1989)

2.13 Toe Protection

The design concept regarding the toe protection of standardized revetments bases on the objective to build all structure components on the dry fluvial plain. In general, the design implies, that the scouring and undermining process of the developing scour hole in front of the structure initiates the deformation process of the toe protection apron (WARPO, 2001). At the estimated maximum scour depth, the falling and launching apron is assumed to cover and stabilize the bank sided river profile, preventing from further erosion of the embankment.

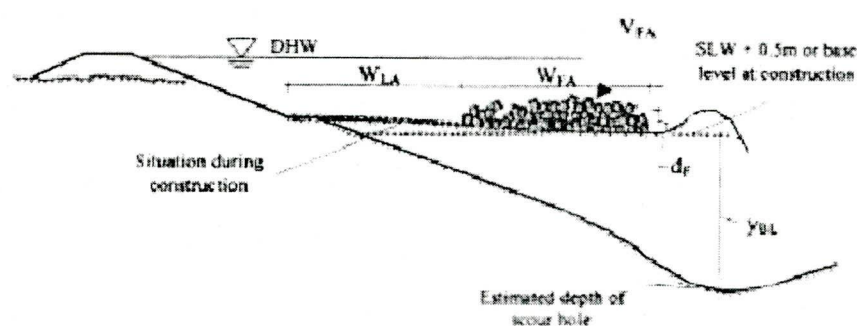


Figure 2.16 Geometric properties of a typical falling apron for standardized revetments (schematic) (Source: WARPO, 2001)

The width of falling apron should be chosen to 70 to 100% of the depth of scour.

Width, $W_{FA} = 0.7$ to $1.0 y_{BL}$

The width of falling apron should be chosen to 70 to 100% of the depth of scour.

Width, $W_{FA} = 0.7$ to $1.0 y_{BL}$

Adequate quantity of stone for the apron has to be provided for ensuring complete protection of the whole of the scoured face.

2.14 Physical Model Study in Bangladesh

In Bangladesh, a Hydraulic Research Laboratory established in 1948 and renamed as "River Research Institute" (RRI) in 1977, has been conducting physical model tests on problems related to hydraulic structures, river training works, and water development projects. Most of these studies have been carried out for Bangladesh Water Development Board (BWDB). They have also conducted a number of model studies for other organizations and private sectors. Among some of the major works

carried out at RRI is the model study of silt excluder tunnel of Teesta Barrage project, carried out in September 1981 (RRI, 1981) to see the design efficiency and hydraulic performance of the silt excluder tunnel system having reference to the discharge in the canal and for exclusion of sand to the canal and to determine the most suitable alignment of the canal head regulator in relation to the axis of the barrage. In order to verify further, the result of model study of an undistorted model of sediment excluder tunnel together with canal head regulator, undersluice and a portion of barrage was constructed and experiments were carried out in June 1982 (RRI, 1982). From the study it was revealed that the canal head regulator with tunnel in three bays with approach guide bank and divide wall would function satisfactorily and would satisfy the design consideration. In 1985, RRI carried out a model study for protection of Kustia town from right bank erosion of Gorai river (RRI, 1985). The study was conducted to find out the most effective and economical measure for protection of Kustia town by constructing groyne/spurs and to determine their location, length, angle alignment etc. of the proposed structure. Recommendation of the model study was that the proposed size and location of the groyne no. 1 would be quite effective and no modification was necessary. Again to retain the newly formed char at the right bank a spur would be constructed. Model study for protection of right flood embankment of the Brahmaputra-Jamuna river conducted by RRI in 1987 was done to find out the method of protecting the Brahmaputra Jamuna right flood embankment from bank erosion of the Brahmaputra-Jamuna river. Study showed that using seven cross bars and three closure at appropriate location may prevent erosion of right bank of the river and these may protect the right flood embankment of Brahmaputra Jamuna river. In 1997, RRI and Jamuna Multipurpose Bridge Authority (JMBA) agreed to execute physical model study of Jamuna river at the Jamuna Multipurpose Bridge Project (JMBP) area at RRI to solve the problems which may arise during the construction and maintenance phases against the hydrodynamic and morphological changes of the river around the bridge site (RRI, 1997). The whole study consisted of two major components e.g. Overall fixed bed model and Movable bed sectional models. The overall model was developed to forecast on overall hydrodynamics and morphology and the sectional models were set up to forecast hydro-morphological changes in the vicinity of the river training works. The models

were found to be effective for hydrodynamic and morphological predictions around the bridge site. Except the above major works there were many other physical model works carried out in RRI for example, a scale model study to find the effectiveness of Multipurpose Groynes as bank protection work to protect Kustia town on Gorai river (Kamal, 1997), scale model study for the protection of Bairagirchar and Chilmari port area from the erosion of Brahmaputra river (RRI, 1996), a scale modelling study of groyne for the protection of Kazipur area from erosion of Jamuna river (Siddique, 1997) etc. that helped the designers and consultants to take necessary steps of their project works. Various model tests conducted at RRI was reviewed by Kashem (1988).

2.15 Mathematical Model Study at the Selected Reach of Old-Brahmaputra River

Surface Water Modelling Centre according to the contract between Roads and Highways Department has carried out a hydraulic mathematical modelling study of the Old Brahmaputra river at Jamalpur-Sherpur-Bongaon Road with the view of (SWMC, 2001):

- Validating (or revising) the hydraulic design parameters for the bridge works
- Providing hydraulic design parameters for the bank protection works
- Providing the impact of the bridge and the bank protection works on the river hydraulics and morphology.

The Old Brahmaputra river is a dynamic meandering river located in the North Central part of Bangladesh. Its offtake is in between Bahadurabad and Chilmari and its downstream end is in the Meghna river near Bhairab Bazar. It is continuously changing its course for decades with erosion/deposition along its banks. Recent successive floods, particularly 1998 and 1999 flood, have accelerated the bank erosion near Jamalpur ferry-ghat. Figure 2.17 shows a satellite image of the Old Brahmaputra showing the study area.

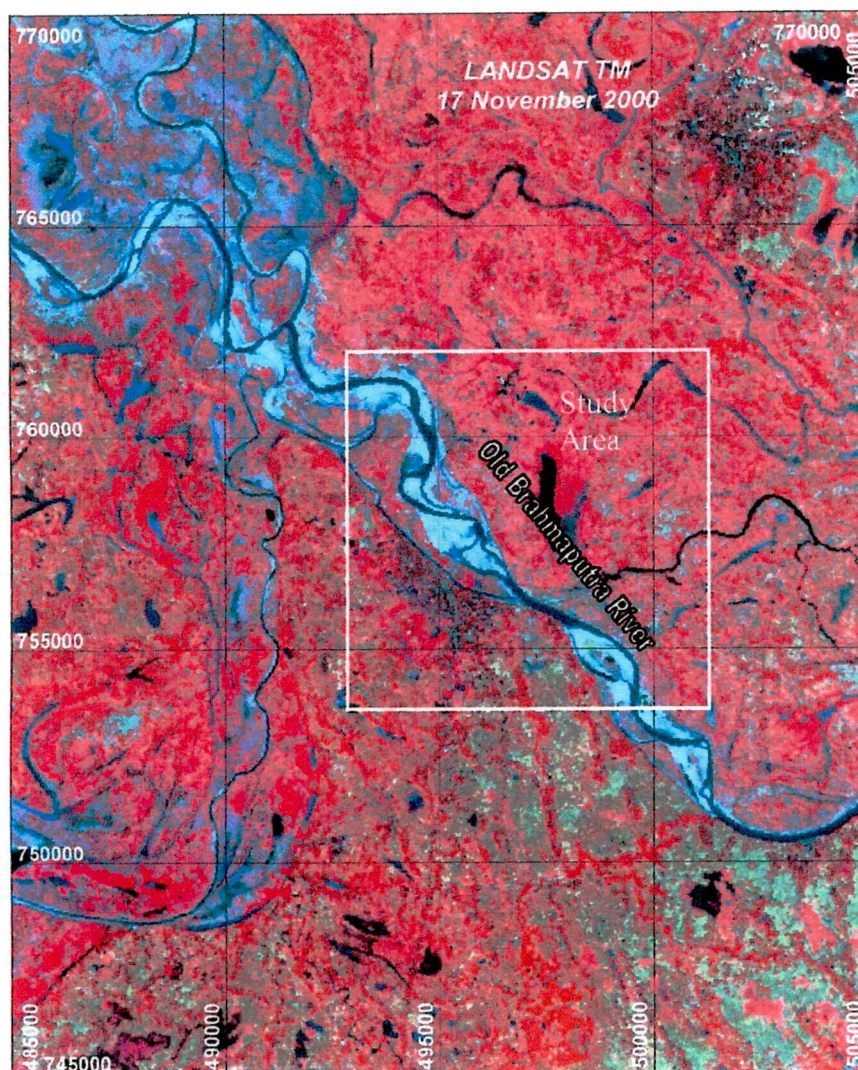


Figure 2.17 Satellite image of the Old Brahmaputra river showing study area
(Source: Institute of Water Modelling)

The model setup is based on the surveyed bathymetric data of May 2000. For the computation of the hydraulic design parameters, the model has been applied for the following design hydrological events:

- 1 in 50 year return period flood event
- 1 in 100 year return period flood event

The model takes into account the impact of the bridge works (bridge piers and approach road on the left bank) and the proposed bank protection work. The model simulation without any project work showed that erosion is generated along the right

bank at the bridge site and its immediate downstream (Figure 2.18). This implies putting more attentions to the right bank in the coming years.

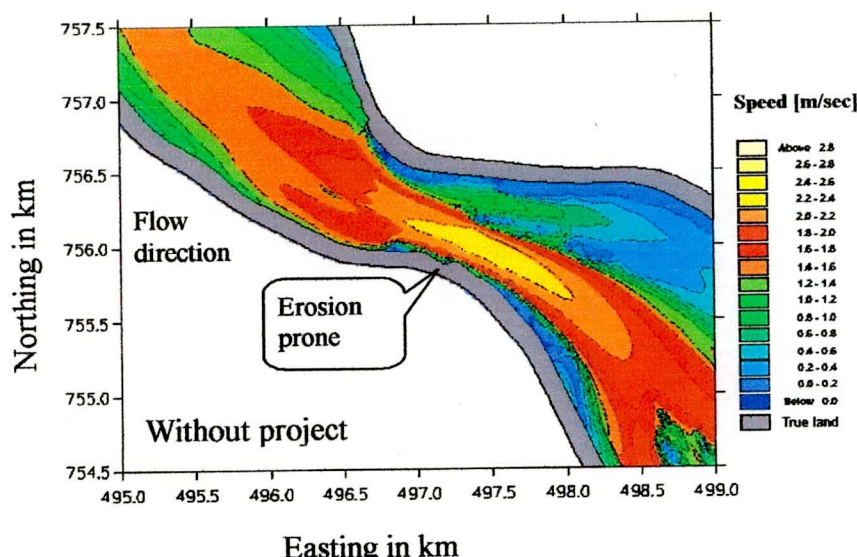


Figure 2.18 Speed contour at the peak of the hydrograph ($Q=6481 \text{ m}^3/\text{s}$) for 1 in 100 year flood event (Source: SWMC, 2001)

The model result considering bridge piers and approach road on the left bank showed that the velocity around the left bank approach road has increased and has developed transverse flow towards the left bank. Bank erosion has developed at its downstream along this bank. The plan form analysis carried out by EGIS II shows that the bridge location is relatively stable than the other places along the river.

CHAPTER THREE

3. DESCRIPTION OF THE PHYSICAL MODEL FACILITIES

3.1 *Introduction*

For long term and short term hydraulic research activities including morphological studies on alluvial channels, a physical model facility has been developed for the Department of Water Resources Engineering, BUET. This model has been used for the study on bank protection by revetment in a reach of the Old Brahmaputra.

3.2 *Location of the physical model*

A 75.5m long and 47.25m wide open space south of Institute of Flood Control and Drainage Research (IFCDR) has been used to develop the physical model facility (Figure 3.1). This model facility is equipped with:

1. Sand Bed
2. Storage Pool
3. Upstream Reservoir
4. Downstream Reservoir
5. Sediment trap
6. Re-circulating Canal
7. Measuring devices
8. Instrument Rails
9. Tail Gates
10. Point Gauges etc.

Since most scale models need protection against wind and rain, shed made of C.I. sheets and truss is planned to be constructed over the study area in future.

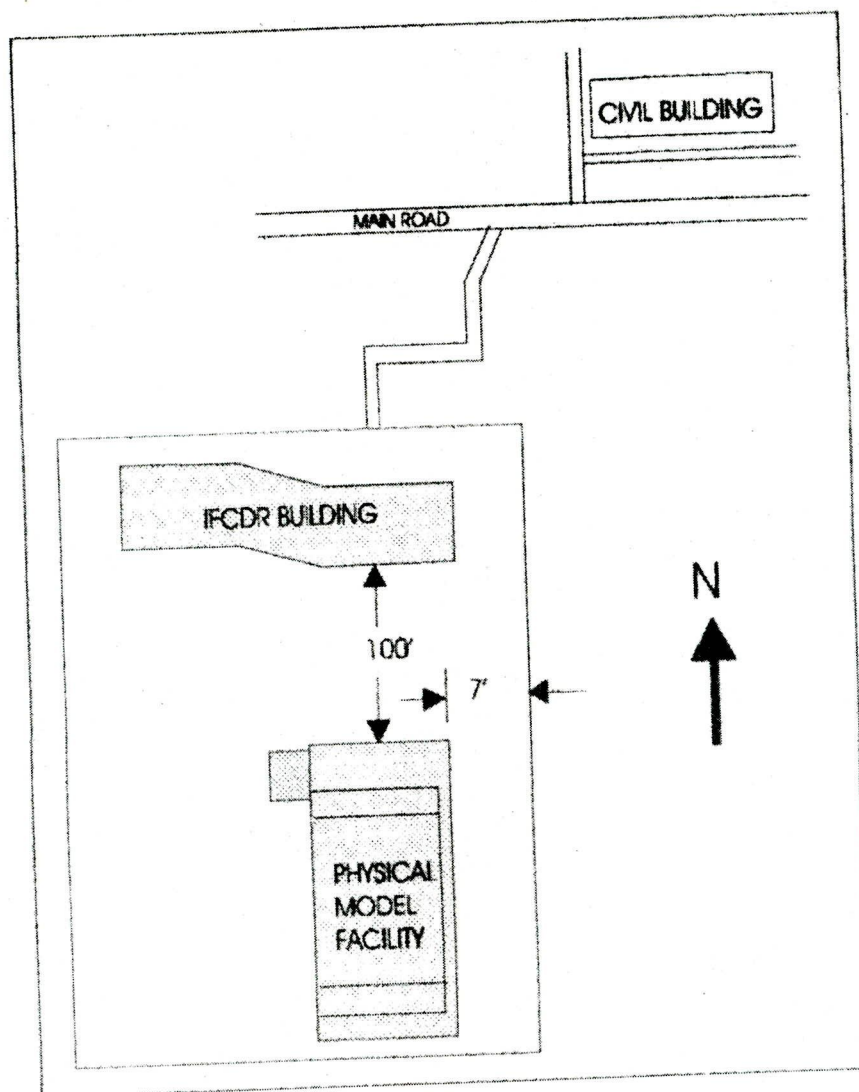


Figure 3.1 Layout Plan of Physical Model Facility (Source: Sarkar S.M.R, 2001)

3.3 *Salient Features of the physical model:*

The salient features of the physical model facility are summarized below (Figure 3.2):

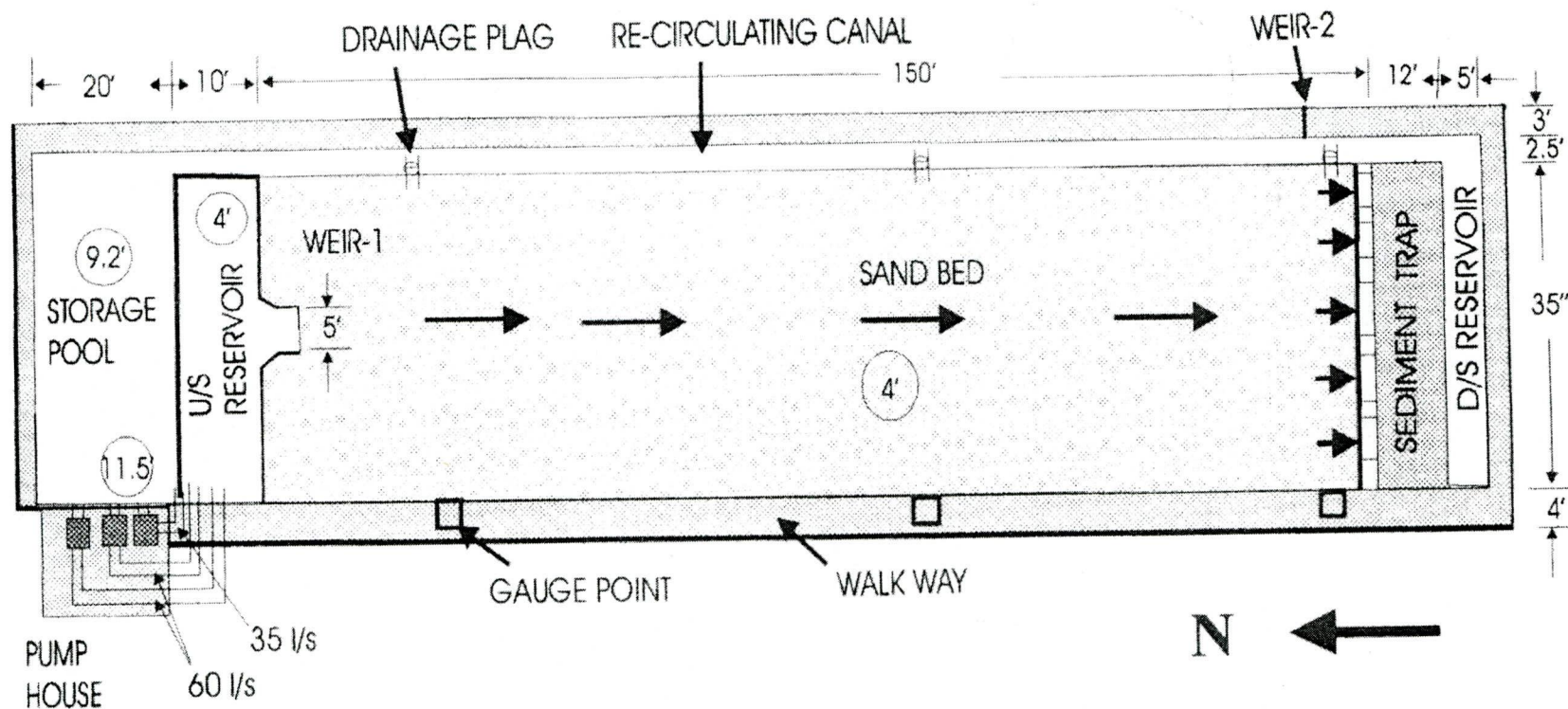


Figure 3.2 Plan View of Physical Model Facility (Source: Sarkar S.M.R, 2001)

3.3.1 Sand Bed

The size of the sand is 45.73m long and 10.67m wide. The depth of the sand bed is 0.91m. The height of the sand bed is 0.61m above the surrounding ground. The sand bed is bounded by 1.22m brick wall on four sides. On the wall at u/s side of the sand bed there is a 1.524m wide Rehbock weir. This weir gives the quantity of water entering into the model. On the wall at the d/s side there are five tailgates for controlling water height in the model. The D_{50} of the sand of the sand bed is 0.18 mm. Two types of sand of sand with $D_{50} = 0.25\text{mm}$ and $D_{50} = 0.1\text{mm}$ has been mixed to get the $D_{50} = 0.18\text{mm}$, which represents the almost similar type of sand of major rivers of Bangladesh. Table 3.1 and Figure 3.3 shows the sieve analysis and gradation chart of a sample from the sand bed. A 0.08m thick brick soling with cement finish was given over a coarse polythene sheet to make the bed leak proof.

Table 3.1: Sieve Analysis of Sample from Sand Bed

Sieve Size	Material Retained	Percent of Material Retained	Cumulative % Retained	Percent Finer	Results
(mm)	gm	%	%	%	$D_{10} = 0.079\text{mm}$ $D_{15} = 0.09\text{mm}$ $D_{50} = 0.18\text{mm}$ $D_{85} = 0.27\text{mm}$
No.4(4.75)	0	0	0	100	
No.8(2.36)	2.2	0	0	100	
No.16(1.18)	1.1	0	1	99	
No.30(0.6)	3.7	1	1	99	
No.40(0.425)	0.8	0	2	98	
No. 50(0.3)	32	6	8	92	
No.100(0.15)	268	54	62	38	
No. 200(0.075)	152.5	31	92	8	
Pan	39.7	8			
Total	500				

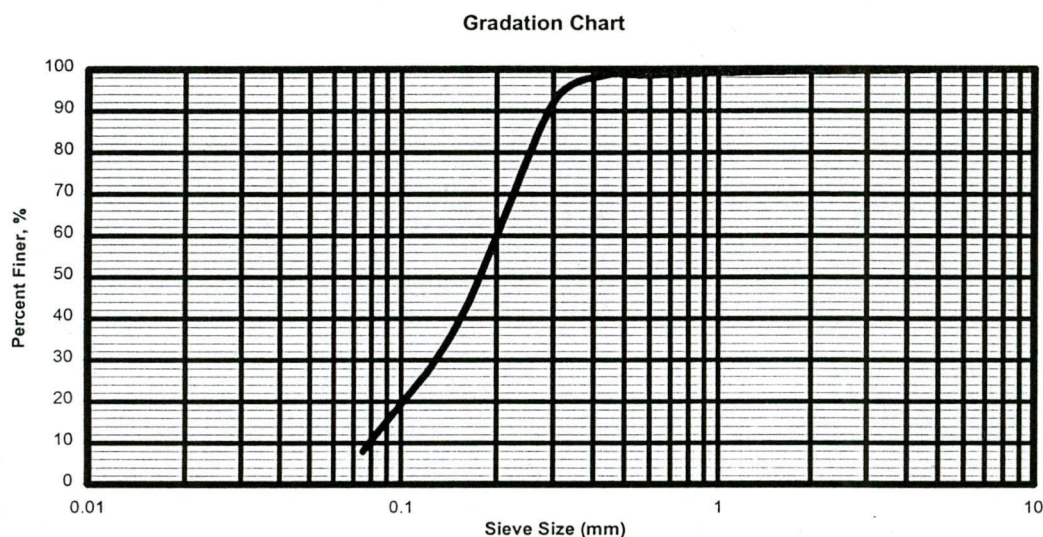


Figure 3.3: Gradation Chart of Sand Bed Material after Sieve Analysis

3.3.2 Storage Pool

The size of the storage pool is 10.67 m x 6.09 m and the depth is 3.2 m. It is the northernmost element at the u/s part of the physical model facility. The total capacity of the pool is 210 m³. The capacity is based on the concept of uninterrupted water supply to the system continuously after the initial saturation of the sand bed and filling up of the other reservoirs. There is an extra 1.22m depth which is provided to accommodate the intake pipes of three pumps.

3.3.3 Pumps

The pump house (Figure 3.2) is located on the western side of the storage pool near the entrance gate. There are three centrifugal electric water pumps installed in this house. The total capacity of the pumps is 220 l/s. Two of the pumps have capacity 80 l/s while the third one is of 60 l/s capacity. However, the total practical capacity was found to be about 150 l/s. The reduction of the capacity may be due to the low voltage or the old age of pumps.

3.3.4 Upstream Reservoir

The upstream reservoir (Figure 3.2) is located between storage pool and the sand bed. There is a 1.524m wide Rehbock weir on the d/s wall. The purpose of this

reservoir is to store water, bring the water body to a steady state and measure the quantity at the Rehbock weir before it is released to the sand bed. The size of this reservoir is 10.67m x 3.048m and the depth is 1.3716m and the capacity is 44.6105m³.

3.3.5 Sediment Trap Tank

The sediment trap tank (Figure 3.2) is located between the sand bed and the d/s reservoir. Its is 1 0.67m x 3.66m with depth of 0.61 m. Considering the sediment size, shape, fall velocity, flow velocity surface area and water depth of the tank the sediment trap tank is designed in such a way that the sediment shall fall to the base due to gravity and clear water gets out of the tank to the d/s reservoir.

3.3.6 Downstream Reservoir

The downstream reservoir (Figure 3.2) is located between the sediment trap at the u/s side and the re-circulating canal at the d/s side. Its size is 10.67 m x 1.524 m and the depth is 0.61 m. The capacity of this reservoir is 9.75 m³/s. The purpose of this reservoir is to store water before it flows to the re-circulating canal. The downstream reservoir is next to the sediment trap tank.

3.3.7 Re-circulating Canal

The re-circulating canal (Figure 3.2) located on the eastern side of sand bed. It connects the downstream reservoir and the storage pool and allows the water to be re-circulated for re-supply. It is 52.44m long, 0.76m wide and 1.28m deep. It is capable of draining all the water supplied to the system. There are three drainage plugs connecting sand bed and the re-circulating canal, which are used for drying up the sand bed after the experiment. There is also a 0.76m weir installed across the re-circulating canal for measuring the quantity of water flowing through the sand bed. The difference of flow between the two weirs is the water loss in the sand bed

3.3.8 Measuring Devices

The measuring devices are the essential part of the physical model facility. These devices control operation of the model facility. A brief summary of these measuring devices are given below:

3.3.8.1 Rehbock Weirs

There are two sharp-crested Rehbock weirs, weirs-1 (Figure 3.4) and weir-2 (Figure 3.5) installed in this facility. Weir-1 is located between u/s reservoir and the sand bed. Its length is 1.524m. It helps to measure the quantity of water flowing to the model by calculating the head from the observation well by point gauge. The Weir-2 is 0.76m long and is installed in the re-circulating canal. The difference of flow between Weir-1 and Weir-2 gives the quantity of water loss in the sand bed.

The discharges of weir 1 (Figure 3.6 and Table 3.2) and weir 2 (Figure 3.7 and Table 3.3) can be found directly from the graphs by knowing the head of water. The difference of levels of weirs and gates are given in Figure 3.8.

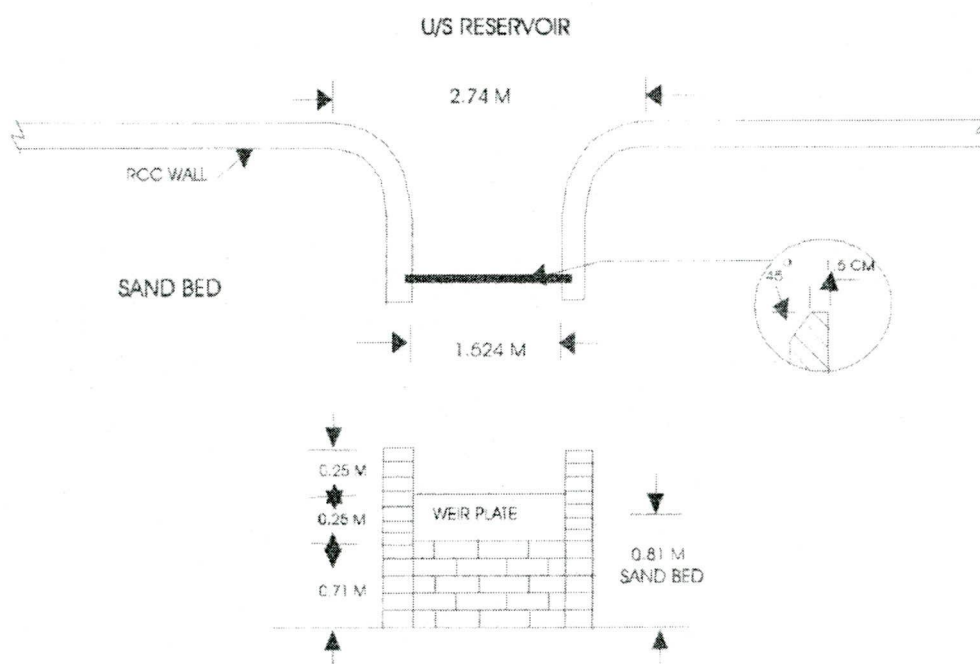


Figure 3.4 Rehbock Weir – 1 (Source: Sarkar S.M.R, 2001)

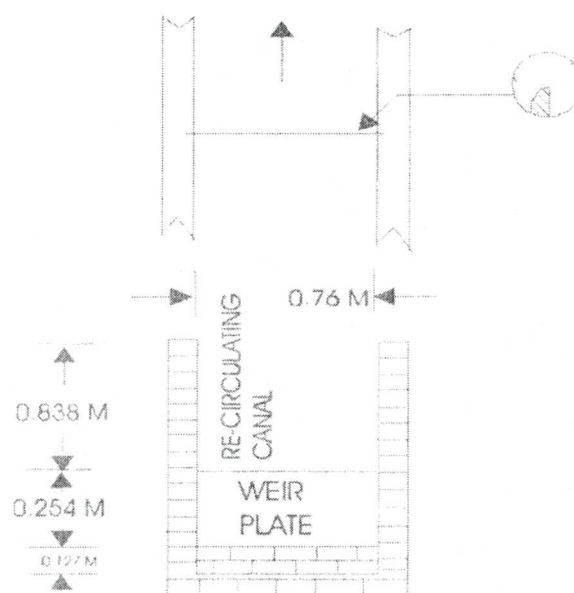


Figure 3.5 Rehbock Weir – 2 (Source: Sarkar S.M.R, 2001)

Table 3.2: Calibration chart of Rehbock Weir-1 at upstream reservoir

Sl. No.	H (m)	H/P	$C_d=0.605+0.001/H+0.08H/P$	$Q_m=2/3C_dBH^{3/2}(2g)^{1/2}$ (l/s)
1	0.01	0.01316	0.70605	3.18
2	0.02	0.02632	0.65711	8.36
3	0.03	0.03947	0.64149	15.00
4	0.04	0.05263	0.63421	22.83
5	0.05	0.06579	0.63026	31.71
6	0.06	0.07895	0.62798	41.54
7	0.07	0.09211	0.62665	52.23
8	0.08	0.10526	0.62592	63.74
9	0.09	0.11842	0.62558	76.01
10	0.10	0.13158	0.62553	89.02
11	0.11	0.14474	0.62567	102.73
12	0.12	0.15789	0.62596	117.10
13	0.13	0.17105	0.62638	132.13
14	0.14	0.18421	0.62688	147.78
15	0.15	0.19737	0.62746	164.05
16	0.16	0.21053	0.62809	180.90
17	0.17	0.22368	0.62878	198.34
18	0.18	0.23684	0.62950	216.35
19	0.19	0.25000	0.63026	234.91
20	0.20	0.26316	0.63105	254.01

where, H = Head over the crest of the weir (Practically measured at a distance 4 to 6 H upstream of the weir crest)

P = Height of the weir crest

C_d = Co-efficient of discharge

B = Width of the weir

g = Acceleration due to gravity

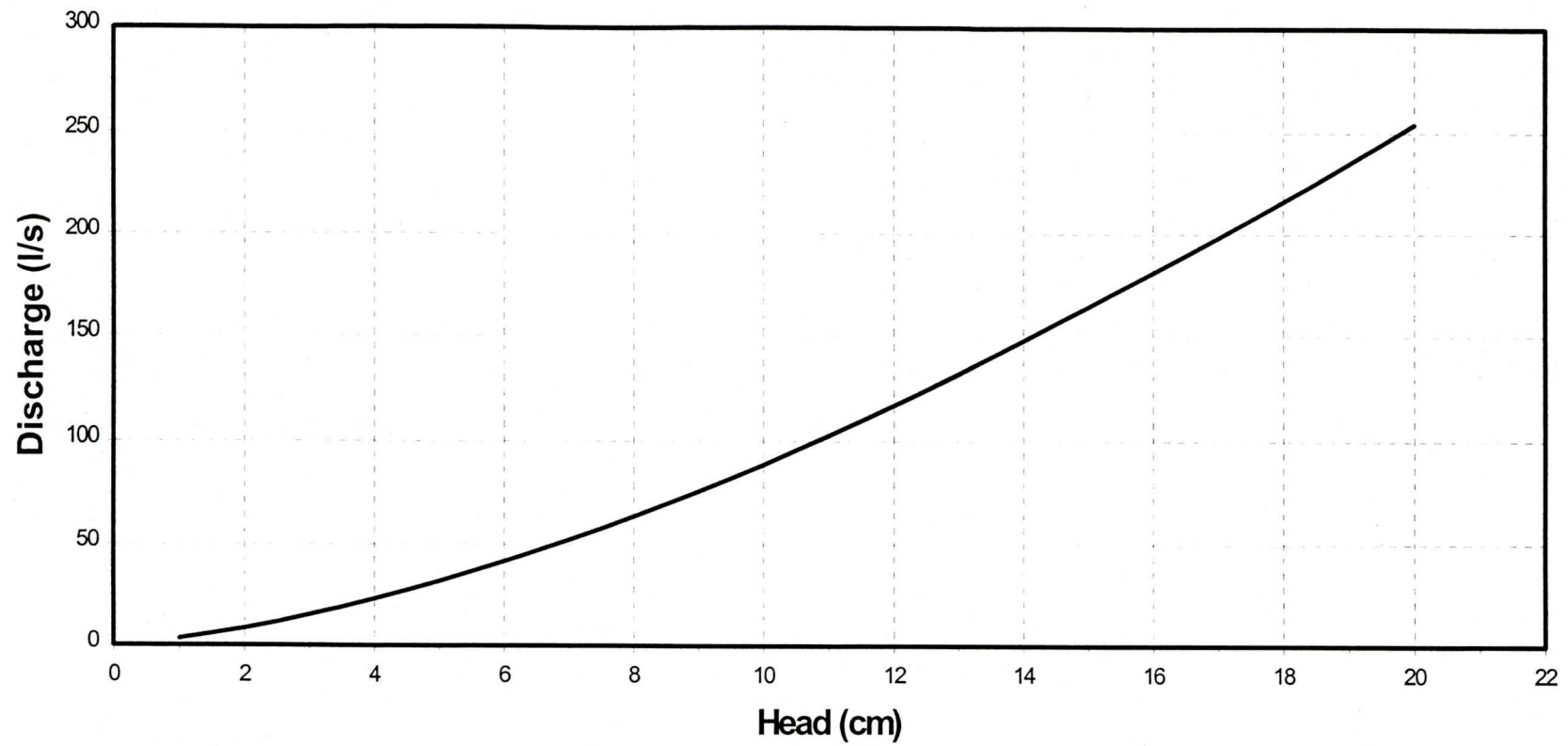


Figure 3.6: Discharge vs. Head Curve of Weir-1

Table 3.3: Calibration chart of Rehbock Weir-2 at upstream reservoir

Sl. No.	H (m)	H/P	$C_d=0.605+0.001/H+0.08H/P$	$Q_m=2/3C_dBH^{3/2}(2g)^{1/2}$ (l/s)
1	0.01	0.02500	0.70700	1.59
2	0.02	0.05000	0.65900	4.18
3	0.03	0.07500	0.64433	7.51
4	0.04	0.10000	0.63800	11.45
5	0.05	0.12500	0.63500	15.93
6	0.06	0.15000	0.63367	20.90
7	0.07	0.17500	0.63329	26.32
8	0.08	0.20000	0.63350	32.17
9	0.09	0.22500	0.63411	38.42
10	0.10	0.25000	0.63500	45.07
11	0.11	0.27500	0.63609	52.08
12	0.12	0.30000	0.63733	59.46
13	0.13	0.32500	0.63869	67.19
14	0.14	0.35000	0.64014	75.26
15	0.15	0.37500	0.64167	83.66
16	0.16	0.40000	0.64325	92.39
17	0.17	0.42500	0.64488	101.44
18	0.18	0.45000	0.64656	110.81
19	0.19	0.47500	0.64826	120.49
20	0.20	0.50000	0.65000	130.48
21	0.21	0.52500	0.65176	140.76
22	0.22	0.55000	0.65355	151.35
23	0.23	0.57500	0.65535	162.23
24	0.24	0.60000	0.65717	173.41
25	0.25	0.62500	0.65900	184.87

Where, H = Head over the crest of the weir (Practically measured at a distance 4 to 6 H upstream of the weir crest)

P = Height of the weir crest

C_d = Co-efficient of discharge

B = Width of the weir

g = Acceleration due to gravity

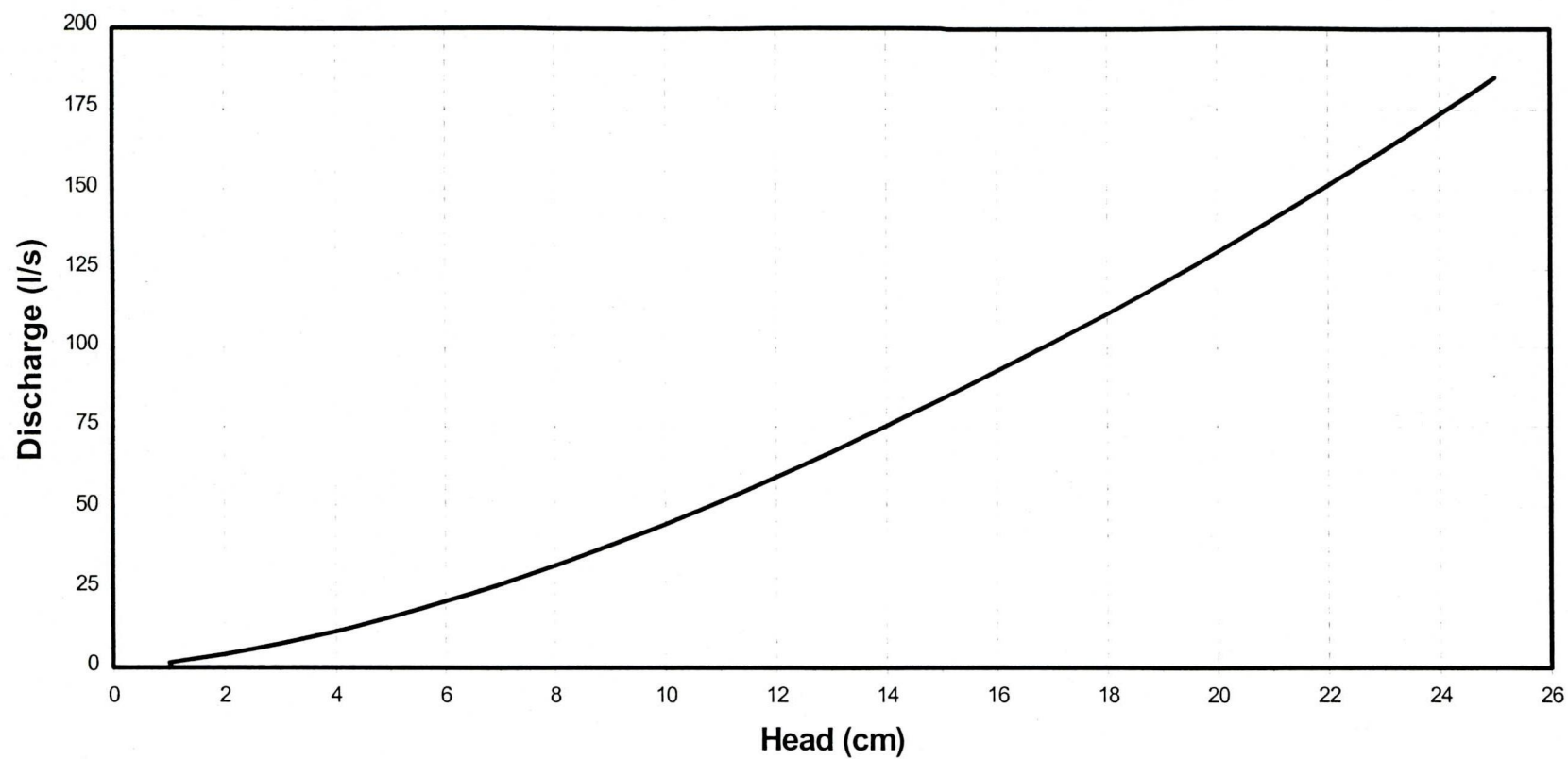


Figure 3.7: Discharge vs. Head Curve of Weir-2

3.3.8.2 Gauge Point

There are three gauge point (Figure 3.2) of the sand bed on the western side. The size of each well is 0.6m x 0.6m and depth is 1.22m. They are connected with the sand bed by 6m long and 0.15m diameter filter pipes (placed at the bottom) to give water level at any time.

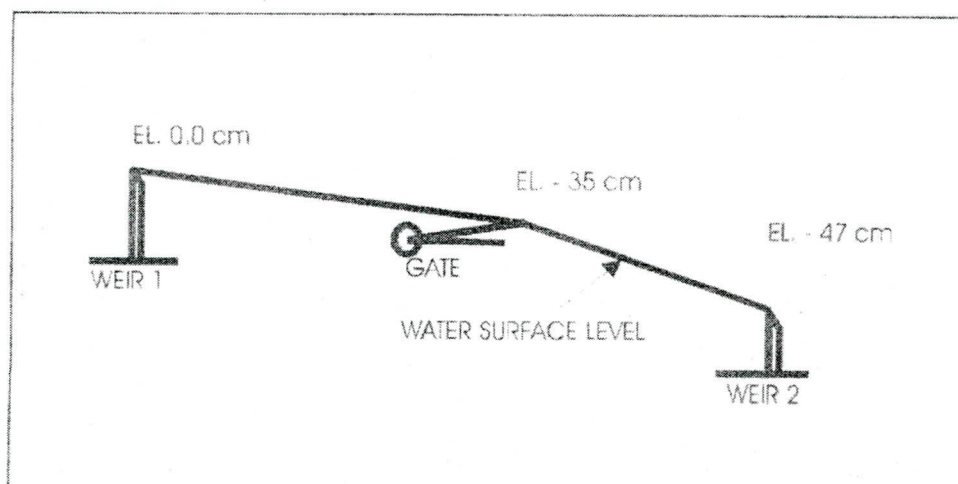


Figure 3.8 Differences of Levels of Measuring Devices (Source: Sarkar S.M.R, 2001)

3.3.8.3 Instrument Rail and Steel Truss Bridge

The steel truss bridge (Figure 3.9) is an essential part for the physical model for taking horizontal and vertical measurements. It is placed over the sand bed and runs over the rails on two sidewalls. Under the truss the gauge can be placed and moved across to take readings of the model bed. There are three benches made to use for walking over the model and take readings without disturbing the bed.

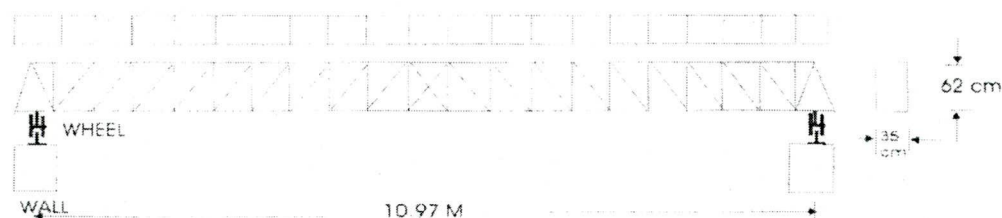


Figure 3.9 Instrument Rail (Source: Sarkar S.M.R, 2001)

3.3.9 Tail Gates

There are total five tailgates placed across the width of the model in between the sand bed and the sediment trap (Figure 3.10). They are provided to maintain the water level of the water in the sand bed as desired. Besides, they prevent the sediment movement out from the sand bed in some percentage. Each gate is made up of 1 mm thick MS plate (0.5 m wide), rubber sheets, cross bar and adjusting rod. The width the middle gate is five feet and remaining four gates are of four feet width. About 0.45m height of water can be manipulated by operating tailgates. There is one gauge point with the middle weir to determine the water height in the model.

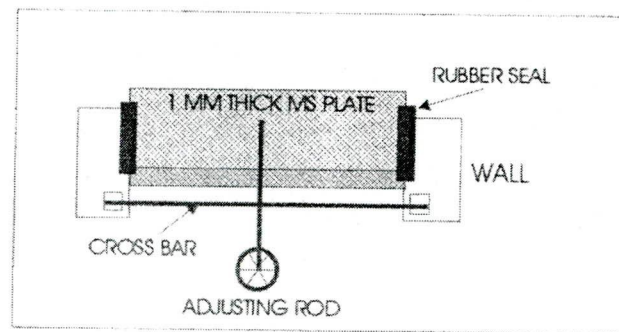


Figure 3.10 Parts of Tail Gate (Source: Sarkar S.M.R, 2001)

3.3.10 Equipment Available in WRE Dept

Following equipments (Table 3.4) are available in the Hydraulic Laboratory of the Department of Water Resources Engineering, which can be used in the physical model facility:

Table 3.4: Equipment Available in Hydraulic Laboratory, BUET

Sl. No.	Name	No of units
1.	Miniature current meter (prototype)	1
2.	programmable 2-D electromagnetic liquid velocity meter	1
3.	Point gauge	6
4.	Pitot tubes and manometers measuring devices, notches, weirs, orifice meters etc.	6
5.	Sediment feeder	2

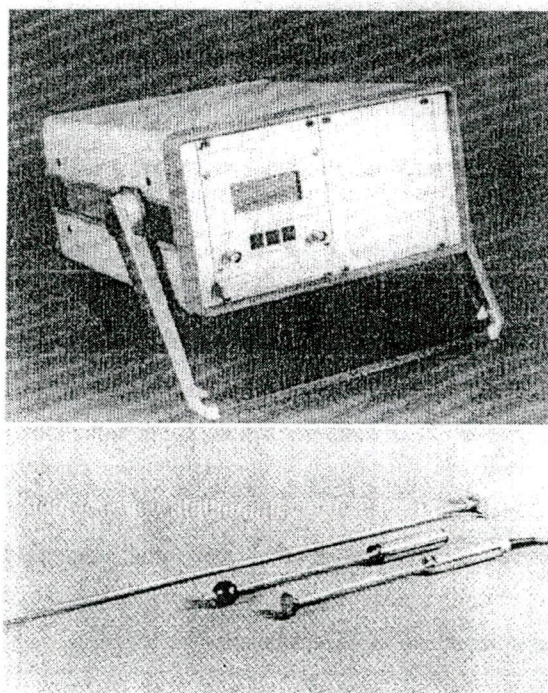


Figure 3.11 Velocity Meter with Probe

3.3.11 Boundary Fence

To prevent any disturbance in the model area from outsiders and wandering animals a boundary fence is essential. Therefore, a wire-mesh fence is constructed around the model on western and northern side of the model area with one gate on the northwest corner near the pump house. This is 1.83m high and 3m away from the model. There is a brick wall on east and southern side.

3.3.12 Walkway

To work efficiently in a physical model, it should be accessible easily from all sides. As such a walk way is constructed all around the physical model by brick soling, the width of the walk way is 1.22m on the west and 0.91m on the remaining sides. It is 0.30m below the top level of the walls of the model. It facilitates easy movement around the model.

3.3.13 Office Facility

The pump house has been constructed for multipurpose use. To facilitate the researchers to do paper-works and to take data by computer by ease a table and some chairs are there. Data recording by using computers can also be done from this house. Measuring tapes, shovels are also available in the pump house.

3.3.14 Shed over the Model

A steel truss shed is to be constructed over the sand bed to protect the model from rain and storm. The design of the shed is completed. It will cover the entire sand bed without any pole on the sand bed to allow the instrument rail to move across freely.

The structural designs of the different parts of the physical model facilities are presented in Appendix A.

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CHAPTER FOUR

4. PREPARATION OF THE SETUP FOR STUDY ON BANK PROTECTION BY REVETMENT

4.1 *Introduction*

In order to prevent or minimize the loss of valuable land, several stretches of the river banks might need suitable protection against erosion. There are several types of bank protection measures practiced in Bangladesh. Among these one of the most popular types of bank protection work is the placing of revetments. Revetment structures are passive bank protection measure which prevent a bank line from erosion but do not create significant interference with the passing water flow. In the present study the performance of cubical block with hexagonal block as river bank protection measure has been evaluated in the physical model for different flow ranges and bank slope conditions. The hexagonal blocks were found to be an efficient coastal bank protection structure. These also enable many cost-saving decisions to be made compared to contemporary units, and can achieve economies in every stage of construction (Brown, 1983). So to analyse its performance as a river bank protection structure and also at the same time to evaluate its performance by comparing it with a common type of revetment i.e. cubical block, used in Bangladesh the following experiments were done on the newly constructed physical model facility.

4.2 *Selection of Model Layout*

To study the performance of different types of revetments as a bank protection structure, a river reach has been selected in such a way that a certain portion of the river bank gets eroded during large flow with high velocity. To ensure such condition a reach from the Old Brahmaputra River in the Jamalpur-Sherpur district line near the Jamalpur ferry-ghat was selected for this study and was scaled down. From a study, conducted by Surface Water Modelling Centre (SWMC, February 2001) it has also been seen that the selected river reach is under severe erosion.

4.3 Scales of Overall Model

The following salient points were considered for the selection of geometric length scale for the present distorted model:

The 15.15 km river reach was scaled down only to get a planform, which will produce erosion on the bank. So the horizontal scale ratio was selected in such a way (1:360) that the meandering nature of the selected river reach is produced. Also the vertical scale ratio (1:25) was selected to keep enough depth in the model river to facilitate the placing of block efficiently and measure the depth and velocity accurately.

During the course of the study the initial bed level for each of the experimental run was kept same. As the actual bed levels of the selected reach of the Old-Brahmaputra river was undulating in nature, it was very hard and time consuming to reproduce those bed levels in the model for every model run. For this reason each of the cross-section was changed into equivalent trapezoidal cross-section and then was reproduced in the model. The plot of actual and equivalent trapezoidal section for different sections of the study reach is presented in Appendix B.

The layout plan of the model area is shown in Figure 4.1. The scale factors for various items in the model are presented in the Table 4.1.

Table 4.1: Scale factors for the model

Parameters	Unit	Scale factor
Length	m	360
Width	m	360
Depth	m	25
Discharge	m ³ /s	45000
Velocity	m/s	5

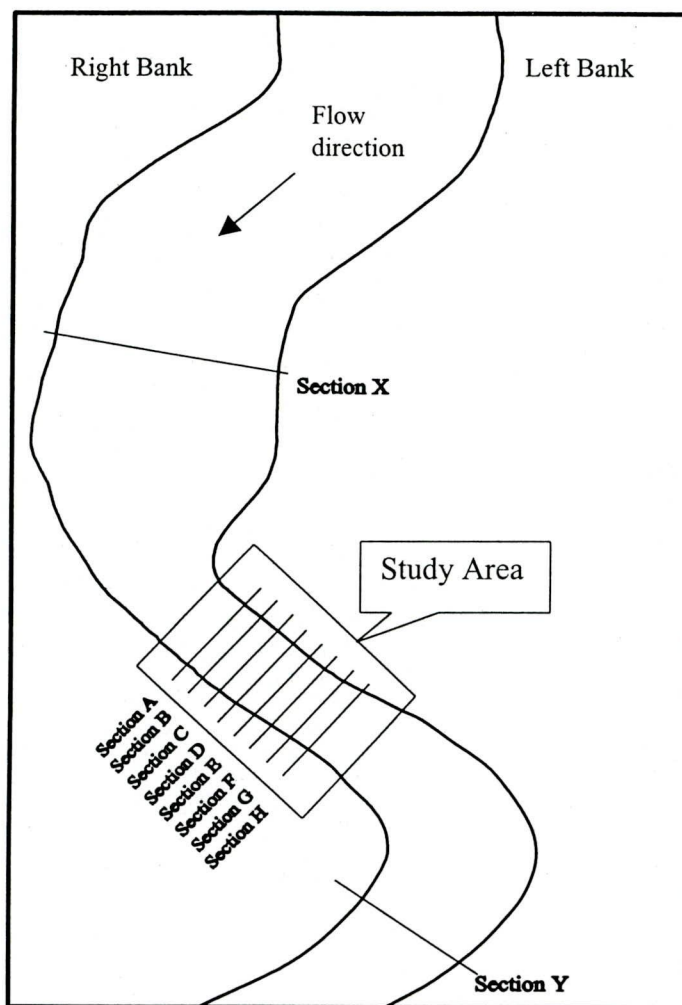


Figure 4.1 Layout plan of the model area

4.4 Model Bed Preparation

The methodology followed for initial model river preparation is discussed as follows:

- The model was carefully inspected and was made free from any unwanted elements.
- The model bed was then levelled with levelling instruments
- To produce the cross-sections of the model river a reference level was needed. So numbers of strings were set across the width of the sand bed as reference level for each cross-section.
- The western wall and eastern wall were taken as reference lines for horizontal measurement. Right bank line and left bank line was first marked by placing

poles after measuring horizontal distance from the western and eastern wall respectively.

- The sand was removed from the bed at each cross-section and the depth of the cross-section was measured vertically from the thread by a scale. By this process the initial model river was formed.
- After this the enter river reach was wetted and the bed and bank of the river was compacted.

The model bed was thus prepared for the initial run without bank protection work.

4.5 Calibration of the Scale Model (Operation of the Model without any bank protection work)

After designing a model with great care, it is essential for distorted as well as undistorted models to be verified against some field data. Through verification of the model, one can understand the degree of representativeness of the model to its prototype.

Following steps were followed at the calibration stage of the Scale Model:

1. The storage pool was filled up with water and the pumps were switched on after priming.
2. Water was then supplied to the model bed from upstream reservoir. Some plastic hollow pipes were fitted upstream of the Rehbock weir of the upstream reservoir to dissipate energy of the flowing water to maintain a uniform flow at the upstream of boundary.
3. The discharge of water was raised very slowly to the model to keep the bed and bank profile undisturbed. After filling up of the model the pumps were controlled to achieve the desired velocity and water depth at the location of interest.
4. The model run was conducted for about 5 hours.
5. It was found that erosion has occurred just at the downstream of outer bend of the model river reach. In the actual field condition erosion has occurred in the same location.
6. After 3.5 to 4 hours velocity measurements were taken at an interval of 20 cm in 8 sections (section A-H) in the longitudinal direction covering the erosion prone area. In the lateral direction measurements were taken at 5 cm interval. In

addition velocity data has been also taken at a u/s section few meters d/s of the u/s boundary (Section X) and another few meters u/s of the d/s boundary (Section Y). A programmable 2-D electromagnetic liquid velocity meter has been used for this purpose. During the experiment tailgates were adjusted manually to maintain desired water level and velocity in the model.

7. Water depth data has also been measured at the above mention sections from the string, the reference level.
8. After the model run the tailgates were lowered slowly and carefully to drain out water. For quick and efficient drainage the drain plugs were kept open. Bed level data were taken on the dry model bed by the help of point gauge on the next day.

The dynamic similarity between the model and the prototype was maintained by keeping the Froudian scale i.e. the Froude number of the model and the Froude number of the prototype, same. This similarity was confirmed by measuring the model velocity and water depth at sections X, Y, A and H and then comparing these with the prototype. Table 4.2 and Table 4.3 show comparisons of the measured velocity and water depth with the observed value of that of prototype.

Table 4.2 Comparison of prototype and model velocity at different sections

Section	Discharge (l/s)	Average velocity from prototype (m/s)	Average velocity from model (m/s)	Percentage of error (%)
Section X	150	0.21	0.199	5.2
Section A		0.39	0.384	1.54
Section H		0.41	0.405	1.22
Section Y		0.26	0.255	2.5

Table 4.3 Comparison of prototype and model water depth at different sections

Section	Discharge (m ³ /s)	Average water depth in prototype (m)	Average water depth from model (m)	Percentage of error (%)
Section X	150	0.22	0.213	3.18
Section A		0.31	0.305	1.61
Section H		0.298	0.29	2.68
Section Y		0.2	0.19	5

4.6 *Scale Model Study with Revetment as Bank Protection Work*

As per definition, revetment is a type of protection for river bank or embankment which covers continuously the entire slope of the bank or the embankment including the portions extending into the river bed to keep the bank from receding landward due to erosion. It is designed to protect and stabilize the slope that may be subjected to action by water current and waves.

There are various types of elements and materials that are used as cover layers including loose boulders, blocks etc. In Bangladesh, for cover layers of standardized revetments, concrete cubes (CC-blocks) are recommended. The blocks should be placed in an orderly manner to provide a uniform weight on the filter-layer and subsoil.

In the present study two types of blocks - Cubical blocks and Hexagonal blocks has been used as cover layer. Granular filter was used as the filter layer between the bank material and the revetments. The performances of each type of revetments were tested on 2H:1V and 3H:1V slopes using granular filter for all cases.

4.6.1 *Dimensioning of the Revetments Blocks*

4.6.1.1 *Dimensioning of Cubical Blocks*

There are number equations available to determine the size of the revetment material to the velocity impinging on it. Some of the most common equations are as follows:

1. Isbash

$$W = \frac{4.1 \times 10^{-5} S_s V^5}{(S_s - 1)^3 \cos^3 \theta} \quad (\text{FPS unit})$$

2. California State Highways (CHS)

$$W = \frac{2 \times 10^{-5} S_s V^6}{(S_s - 1)^3 \sin^3 (70 - \theta)} \quad (\text{FPS unit})$$

3. Neill

$$d = 0.034 V^2 \quad (\text{Metric unit})$$

4. PIANC

$$d_n = \frac{0.7V^2}{g(S_s - 1)\cos\theta\left(1 - \frac{\tan^2\theta}{\tan^2\phi}\right)^{0.5}} \quad (\text{FPS or metric unit})$$

5. JMBA

$$d_n = \frac{0.7V^2}{g(S_s - 1)\left[\log\left(\frac{6h}{d}\right)\right]^2\left[1 - \left(\frac{\sin\theta}{\sin\phi}\right)^2\right]^{0.5}} \quad (\text{FPS or metric unit})$$

where, in the above equations,

W = Weight of individual stone (lb. wt.)

d = Diameter of stone (ft or m)

d_n = Dimension of cube (ft or m)

V = Velocity (ft/s or m/s)

h = Depth of water (ft or m)

S_s = Specific gravity of stone

θ = Slope of bank

ϕ = Angle of repose of revetment materials

Ψ = Shields constant

g = Gravitational acceleration (ft/s² or m/s²)

6. Pilarczyk

The general formula for the design against current loads is given by,

$$D_n \geq \frac{0.035 \bar{u}^2}{\Delta_m \cdot 2g} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{K_S \cdot \Psi_{cr}} \quad (\text{Metric unit})$$

where,

D_n (m) = characteristic size of the revetment cover layer material (single unit size for loose elements, thickness of mattress system)

$\bar{u}$ (m/s) = depth averaged flow velocity

Δ_m = relative density of submerged material = $(\rho_s - \rho_w) / \rho_w$

g (m/s²) = acceleration due to gravity

ϕ_{SC} = stability factor for current

Ψ_{cr} = critical shear stress parameter

K_t = turbulence factor

K_h = depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio:

$$K_h = 2. \left[\log \left(\frac{12h}{k_r} \right) \right]^{-2} \text{ with } k_r = D_n \text{ for relatively smooth material}$$

$$K_s = \text{bank normal slope factor } K_s = K_s = \sqrt{1 - \left(\frac{\sin \alpha}{\sin \varepsilon_s} \right)^2}$$

α ($^\circ$) = slope angle of bank or structure

ε_s ($^\circ$) = angle of repose considering the material specific internal friction

For this study, several equations were used to find the dimensions of cubical revetment. And, after applying a factor of safety of 1.2 the final dimension of the cube was found. Table 4.4 gives the dimensions of cubical block from different equations:

Table 4.4 Dimensions of cubical block from different equations

Sl. No.	Equation	Dimension (cm)
1.	Neill	1.02
2.	Pilarczyk	0.96

Finally selected dimensions of the cubical blocks are as follows:

Thickness: 1 cm

Length of each side: 1 cm

Surface area of each face: 1 cm²

Volume: 1 cm³

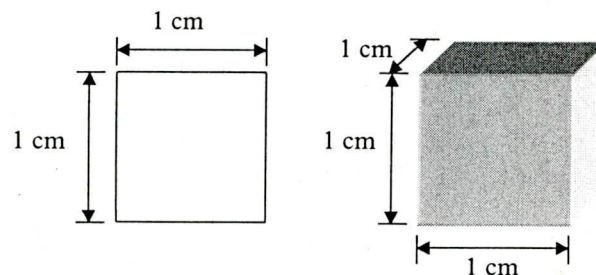


Figure 4.2 Dimensions of Cubical Blocks

4.6.1.2 Dimensioning of Hexagonal Blocks

The mass and thickness of hexagonal blocks when they are used as coastal structures can be found from the following formulas (C. T. Brown, 1983):

$$t = H/(1-p)C_B\phi(a)(s-1) \quad \text{Eq. 4.1}$$

$$M = R.A_g(1-p)S \quad \text{Eq. 4.2}$$

where, t = Armour layer thickness

H = Design wave height

p = armour layer porosity

C_B = Hydraulic performance coefficient of the layer

$\phi(a)$ = an angle function approx. = $\cot\alpha^{1/3}$

s = relative density of the armour unit

M = Mass of the armour unit

The thickness of the revetment was calculated by using the above equation for Hexagonal armour unit under wave attack, where wave height has been determined from the nomographs of deepwater significant wave prediction curves as a function of wind speed, fetch length, and wind duration. From the above equation the thickness of the hexagonal revetment under current attack was found to be 5 mm. In open channels revetment protection is subjected to hydro-dynamic drag and lift forces that tend to erode the revetment and reduce its stability. The drag and lift forces are created by flow velocities adjacent to the revetments. Forces resisting motion are the submerged weight of the revetment and any downward and lateral force components caused by contact with other revetments in the protection layer. So to keep the two types of blocks comparable the volume as well as the weight (as same material is used) of the two types of blocks were kept same.

Finally selected dimensions of the hexagonal blocks are as follows:

Thickness: 5 mm

Length of each of the 6 sides: 9 mm

Surface area of top and bottom face: 2 cm^2

Volume: 1 cm^3

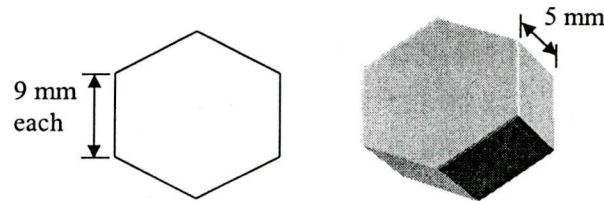


Figure 4.3 Dimensions of Hexagonal Blocks

4.6.1.3 *Reproduction of the Prototype Block Dimensions*

To get the prototype dimensions of the blocks, the model velocity that was used to design the model blocks was converted into prototype velocity using the scale factor for velocity. This velocity (2.5 m/s) was then used to calculate prototype block dimensions using the same formulae used for model. The dimensions of the calculated prototype dimensions are given in Table 4.5.

Table 4.5: Dimensions of blocks in prototype

Dimensions	For Cubical Block (mm)	For Hexagonal Block (mm)
Length of each side of the surface area	260	230
Thickness	260	125

The above results show that the model results can be used in the field condition. But in case of hexagonal blocks it is better to do further experiments using these blocks to understand their behavior in different situations before applying it in the actual field.

4.6.1.4 *Preliminary Design Guide Lines for Hexagonal Blocks*

The above methodology used to determine the dimension of hexagonal blocks can be used as the initial design guideline for further research work in the future. Significant wave height can be determined from the nomograms of deepwater significant wave prediction curves as a function of wind speed, fetch length, and wind duration. Then using Equation 4.1 the required thickness of the hexagonal block can be found. Volume of the block can be kept equal to the volume of cubical block found from

design. But before practical use more experimental runs with different hexagonal block size on different bank slope and flow condition should be done.

4.6.2 Construction of Revetment Blocks

The blocks were made using the cement: sand: water ratio of 1:2:0.5. Each of the blocks was prepared with great care. Special metal frames were made to construct the hexagonal revetments. The blocks were kept in water for curing. A total of 20,000 cubical blocks and 10,000 hexagonal blocks were constructed which has been used to cover a $77,000\text{cm}^2$ surface area of the erosion prone area of the bank. It is worth mentioning here that to cover the same area the material required for the construction of hexagonal blocks were just half of that was necessary for cubical blocks.

4.6.3 Design of Filter

The stability of the whole revetment depends strongly on the filter layer. The filter layer has to separate the cover layer from the subsoil, to provide a drainage zone parallel to the slope of the revetment to protect the subsoil from erosion when flowing over its surface parallel to the slope and to dissipate the energy of the internal flow in the subsoil caused by wave or current action.

In the present study granular filter has been used to keep the subsoil particles from invading the filter, the success of which depends on particle size. The filter criterion relates the grading of the filter to that of the subsoil. It is also necessary to check for filter uniformity to ensure that internal migration of filter material does not occur. Therefore granular filters have been designed in successively coarse layers proceeding outward from the underlying finest soil. The first layer was to hold the subsoil, while each following layer was to be able to hold the underlying one.

The following table provides the designed granular filter specifications using different methods.

Table 4.6 Design Specification of filter material using different method

Method used	Specifications
BWDB	$d_{15} \leq 1.35 \text{ mm}$ $d_{50} < (1.8 \text{ mm} - 5.4 \text{ mm})$ $d_{15} \leq (0.54 \text{ mm} - 1.8 \text{ mm})$ Minimum layer thickness = 4 mm
US Bureau of Reclamation	$d_{50} < 0.9 \text{ mm} - 1.8 \text{ mm}$ $U < 5$
US Crop of Engineers	$d_{15} < 1.35 \text{ mm}$ $d_{15} < 1.8 \text{ mm}$ $d_{50} < 4.5 \text{ mm}$
Pilarezyk	$d_{15} < (1.08 \text{ mm} - 1.35 \text{ mm})$ (stability criterion) $d_{15} > (0.36 \text{ mm} - 0.45 \text{ mm})$ (permeability criterion) $d_{50} < (3.6 \text{ mm} - 4.5 \text{ mm})$ $U < 10$ (for no migration)

Considering the above calculations, finally selected granular filter specifications are:

$$d_{15} = 1.25 \text{ mm}$$

$$d_{50} = 3 \text{ mm}$$

$$\text{Thickness} = 4 \text{ mm}$$

Table 4.7 and Figure 4.4 shows the sieve analysis and gradation chart of the sample from the filter material.

Table 4.7: Sieve Analysis of Sample from Filter Material

Sieve Size	Material Retained	Percent of Material Retained	Cumulative % Retained	Percent Finer	Results
(mm)	gm	%	%	%	
No.3 (6.35)	0	0	0	100	$D_{10} = 0.98\text{mm}$ $D_{15} = 1.25\text{mm}$ $D_{50} = 3.00\text{mm}$ $D_{60} = 3.35\text{mm}$
No.4 (4.75)	0	0	0	100	
No.8 (2.36)	650	65	65	35	
No.16 (1.18)	210	21	86	14	
No.30 (0.6)	90	9	95	5	
Pan	50				
Total	1000				

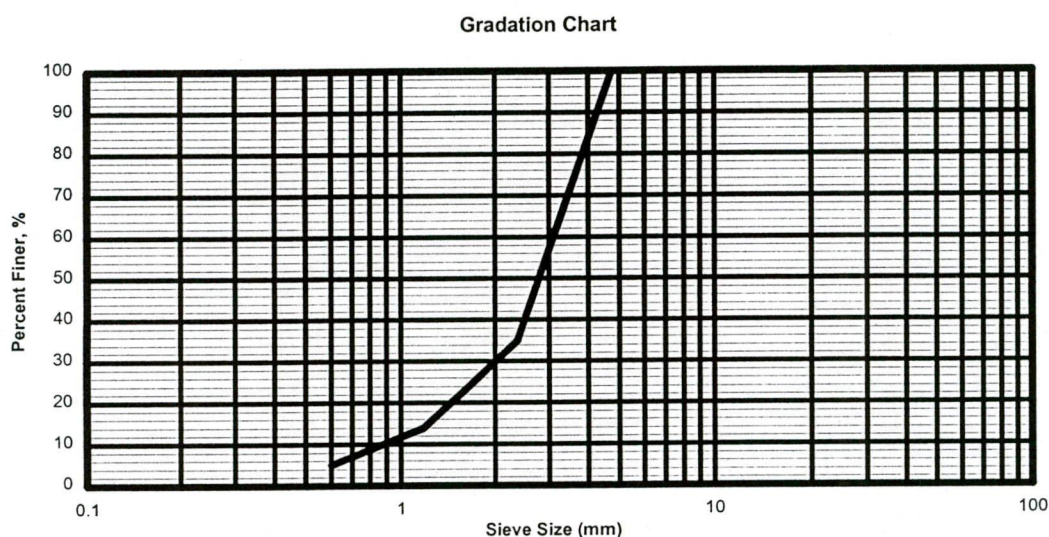


Figure 4.4 Gradation Chart of Filter Material after Sieve Analysis

4.6.4 Design of Toe Protection

Toe protection is required where currents and/or waves scour and undermine the toe of a bank or an embankment, which are likely to result in sliding of the slope, thus endangering the overall stability and function of the revetment.

4.6.4.1 Effective volume and dimensions of falling apron

To calculate the effective dimensions of falling apron the expected scour depth has been calculated as following:

$$R = \text{Lacey's normal scoured depth} = 0.47(Q/f)^{1/3}$$

$$\text{Where, } f = \text{silt factor} = 1.76\sqrt{d_{50}} = 0.7467$$

$$Q = \text{Discharge} = 200 \text{ l/s}$$

$$\text{So, } R = 30.3 \text{ cm}$$

Table 4.8 Formulas for scour depth calculation

Sl. No.	Location	Mean values of x	$y_{BL} = xR - \text{water depth above bed}(y)$
1.	Noses of guide banks	2.25	$2.25R - y$
2.	Transitions from noses to straight portion	1.50	$1.5R - y$
3.	Straight reaches of guide bunds	1.25	$1.25R - y$

Average water depth above model bed, $y = 30$ cm

y_{BL} (at transition) $= 1.5R - y = 15.45$ cm, and

y_{BL} (at straight reach) $= 1.25R - y = 7.88$ cm

The width of falling apron should be chosen to 70 to 100% of the depth of scour.

Width, $W_{FA} = 0.7$ to $1.0 y_{BL}$

$$\therefore W_{FA} = 0.8 \times 0.15 = 10 \text{ cm}$$

The theoretical thickness of the falling apron d_{FA} (without voids) has been calculated from,

$$d_{FA} = 5.0 \text{ to } 8.5 D_n$$

$$d_{FA} = 5.0 \times 0.01 = 0.05 \text{ m} = 5 \text{ cm (without voids)}$$

The blocks and equal sized stones were used as falling apron. A sufficient reserve of stones were kept nearby so that it can be used in time of need.

4.6.5 Placement of Filter and Revetment Blocks

Earth filling and compaction of the loose earth was done to achieve the desired slope of the bank of the study area. The filter materials were then laid on the smoothed bank carefully so that a uniform thickness of the filter layer is maintained all over the protection area. The placing of revetment blocks on the slope bank has been started over the filter layer after the placement of toe protection of the revetment has been laid properly. Each of the concrete blocks has been laid by hand on the filter layer in rows parallel to the direction of the flow starting at the toe of the slope in horizontal layers working upwards. The blocks in each row were staggered half a block width from those in the previous rows. While placing, minimum of voids, but a maximum of interlocking of the revetment has been ensured. It is important to note that the placement time required for hexagonal blocks was almost half of that required for the cubical blocks. The upstream and downstream ends of the protection layer was protected against erosion by keying the two ends into the banks and placing stones with size equal to the cubical blocks over the these transition zones.

4.7 Experimental Runs

The experimental runs have been conducted on 2H:1V and 3H:1V slope with Cubical and Hexagonal blocks as revetment for different velocity ranges within the study area. A brief summary of the model runs that have been done are given in Table 4.9.

Table 4.9 Details of Experimental Runs

Revetment Type	Slope	Average Velocity (cm/sec)			
		50	60	70	80
Cubical Block	1V:2H	√	√	√	√
	1V:3H	√	√	√	√
Hexagonal Block	1V:2H	√	√	√	√
	1V:3H	√	√	√	√

Photographs which are taken during experiment are presented in Appendix C. After each model run (i.e. for a particular type of revetment with particular slope at different velocities) the entire model river was again reconstructed as it is necessary to bring it in its initial measurements and condition. Thus in each run the initial condition of the bed was kept the same. The erosion prone area, where the bank protection has been applied, is the only part of the river reach that was changed in different runs.

CHAPTER FIVE

5. ANALYSIS OF EXPERIMENTAL RESULTS AND DISCUSSION

5.1 *Introduction*

In river engineering it is often required to stabilize eroding river-banks against shifting streams. This may be to prevent loss of land or to protect river structures. Wide varieties of protective works can be taken to control the river but nowadays the most widely used material is CC-block type revetment in Bangladesh. In this study, bank protection by two different types of revetment against current attack was studied. In addition, two types of revetment placement were also examined, where the effects of the shape of revetment cover, two different bank slopes at different flow velocities, on the failure of revetment were analysed.

5.2 *Analysis of Experimental Results*

The main subject of this study is to determine the failure of revetment structure. For this purpose 16 runs were performed. In these runs, two different shapes of revetment e.g. Cubical and Hexagonal type blocks, two different side slopes for placement of these revetments, i.e. 2:1 and 3:1 for each type of revetment at 50 – 80 cm/sec ranges of flow velocities were changed to see the effects of these parameters on the failure of revetment.

In this study, two types of measurement have been taken. Those are:

- Measurement of flow velocity (Adjacent to the revetment structures and also average flow velocity in the study section), and
- Measurement of bank profile of revetment structure in the protected area.

All measurements are taken within the study area, which is protected by revetment works.

5.2.1 *Measurement of Flow Velocity*

Velocity measurements near the bank should be used rather than average velocities through the channel cross-section, for the failure of revetment. Because the revetment materials are under influence of the velocities adjacent to revetment, not the average velocity of flow and at the other hand velocity distribution through the

cross-section is not uniform. Moreover, the appropriate velocity for use in the riprap design procedure must be determined. The velocity used must be representative of flow conditions at the riprap and must be able to be determined by the designer by relatively simple methods. Average channel velocity can usually be determined but is not representative of conditions at the concrete surface because of the wide diversity of channel shapes. Some form of local velocity must be used to account for the differences in channel and revetment shapes. Local bottom velocity is the most representative velocity but is difficult for the designer to predict. Therefore, local average velocity at the toe of the structure, also called local depth-averaged velocity (Maynard et al. 1989), is the velocity used in this study. Local average velocity is representative of flow conditions at the point of interest and can be estimated by the designer.

Therefore, in this study, local velocity measurements were taken at different sections of the study area along the revetment structure and also the average flow velocity of the sections. These velocities help to determine the relationship between the local depth-averaged velocities at the toe of the structure with the average flow velocities. In addition, using these velocities, the effects of channel side slopes and channel bends on stability of the different types of revetment structures were also studied. Moreover, pattern of flow velocities along the side slope of the structure at different sections of the protected area in different run scenarios were also observed.

5.2.2 Analysis of Velocity Data

To compare the stability of the above-mentioned two different types of revetment structures at velocity point of view, local depth-averaged velocity has been analysed for use in the low turbulent environment found in most open channels. Moreover, to show a relationship between the local depth-averaged velocity in the bend over the toe of the side slope and average channel velocity, measured velocities were analysed.

5.2.2.1 Relationship Between Local Depth-Averaged and Channel Average Velocity

To use the local depth-averaged velocity instead of channel average velocity for the design of revetment structure and stability analysis, a relationship between these two velocities have been developed and presented in Table 5.1. Using these relationships, designer will be able to estimate the local depth-averaged velocity at the point of interest. Estimation of local depth-averaged velocity in a physical model is a laborious job, which requires the measurement of local velocities along the side slope of the revetment structure in a grided manner both vertically and horizontally. It also requires the measurement of average channel velocities. After the measurements of the local velocities at different grid points, local depth-averaged velocities can be obtained at the point of interest. Finally, ratio between these two velocities will reveal a relationship between the two velocities. Therefore, the relationship can be presented in the following form:

$$V_{ss} = CV_{avg}$$

where, V_{ss} = local depth-averaged velocity in the bend over the point of interest at the side slope; V_{avg} = average channel velocity; and C = is the coefficient to be determined to establish the relationship from physical model study or by any other measurement techniques.

Table 5.1 Relationships between Velocities

Average Channel Velocity, V_{avg} (cm/s)	Local Depth-Averaged Velocity, V_{ss} (cm/s)*			
	For Cubical Type Blocks		For Hexagonal Type Blocks	
	2H:1V	3H:1V	2H:1V	3H:1V
50	57.5	57.6	57.7	58.0
60	68.7	69.4	69.6	69.9
70	80.5	81.3	81.7	82.4
80	92.0	93.0	93.5	94.5

* These velocities were measured adjacent to the toe of the side slope

5.2.2.2 Stability Analysis using Local Depth-Averaged Velocity Data

5.2.2.2.1 Dimensional Analysis

The dimensional analysis follows the analyses previously presented by Neill (1967). The pertinent variables applicable to the stability of coarse particles

$$f(d, D, \rho, V, \gamma'_s, \nu) = 0 \quad \text{Eq. 5.1}$$

where, d = characteristic particle size; D = depth; ρ = water density; V = characteristic velocity; γ'_s = submerged specific weight of particle; and ν = kinematic viscosity. γ'_s can be defined by the expression $g(\rho_s - \rho)$ where g is the gravitational acceleration and ρ_s is the particle density. ν can be defined by the expression μ/ρ where μ is the absolute viscosity. Neill used the repeating variables V , d , and γ'_s . By requiring rough turbulent flow, the viscosity effects are eliminated and the dimensionless ratios are found to relate in the following form

$$\frac{V^2}{\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)gd} = f\left(\frac{d}{D}\right) \quad \text{Eq. 5.2}$$

where, γ_s = specific weight of stone = $g\rho_s$; and γ_w = specific weight of water = $g\rho$. In this study, the pertinent variables are as shown in Eq. 5.1 with the repeating variables of V , D , and γ'_s . By requiring rough turbulent flow the viscosity effects are eliminated and the dimensionless ratios are found to relate in the following form

$$\frac{d}{D} = f\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5} \frac{V}{\sqrt{gD}}\right] \quad \text{Eq. 5.3}$$

Eq. 5.3 is presented in the same form in Bogardi (1978). The right side of Eq. 5.3 contains a local Froude number. Almost all riprap problems concern subcritical flow and that reduces the significance of defining this as a Froude number.

Using Eq. 5.2, Neill's (1967) evaluation of experimental data defining first movement of a coarse, uniform material resulted in

$$\frac{V^2}{\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)gd} = C_1 \left(\frac{d}{D}\right)^{-0.2} \quad \text{Eq. 5.4}$$

where, C_I is a coefficient. The coefficient subscript is used to differentiate coefficients. Using Eq. 5.3, the analysis of experimental data defining riprap stability presented by Maynard (1988) resulted in the equation

$$\frac{d}{D} = C_2 \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V}{\sqrt{gD}} \right]^{2.5} \quad \text{Eq. 5.5}$$

Except for the coefficients. Eqs. 5.4 and 5.5 are identical.

5.2.2.2.2 Comparison with Existing Riprap Sizing Relations

Three commonly used methods of riprap design are the tractive force with a constant Shields coefficient, the Ishash (1935) and the Neill (1975) type relation. Eq. 5.4 and 5.5 are comparable with these three methods as follows:

Tractive Force Relations

The tractive force relations are

$$T = \gamma_w DS \quad \text{Eq. 5.6}$$

and

$$T_c = C_c (\gamma_s \gamma_w) d \quad \text{Eq. 5.7}$$

where T = tractive force imposed on the boundary; T_c = critical tractive force for size d ; C_c = Shields' coefficient; and S = slope. The Manning and Strickler equations are

$$V = \frac{1.49}{n} D^{2/3} S^{1/2} \quad \text{Eq. 5.8}$$

and

$$n = C_3 d^{1/6} \quad \text{Eq. 5.9}$$

where n is the Manning's resistance coefficient and C_3 is a coefficient. By assuming that the numerator of the Manning's equation contains $\sqrt{g}$, Equation 5.6 – 5.9 can be combined into the relation

$$\frac{d}{D} = C_4 \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V}{\sqrt{gD}} \right]^3 \quad \text{Eq. 5.10}$$

Equation 5.10 has been used by Straub (1953), Grace et al. (1973), Maynard (1978), and Reese (1984).

Isbash Relation

The Isbash relation is

$$d = c_5 V^2 \quad \text{Eq. 5.11}$$

Neill Relation

The Neill relation is

$$d = c_6 V^2 \quad \text{Eq. 5.12}$$

The Isbash and Neill relation, which does not depend on depth, can be rewritten in the following form where depth cancels out from each side of the equation

$$\frac{d}{D} = C_7 \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V}{\sqrt{gD}} \right]^2 \quad \text{Eq. 5.13}$$

In this study, Equation 5.13 is used for stability comparison of different types of revetments using local depth-averaged velocity data.

5.2.2.2.3 Effects of Channel Side Slopes and Channel Bends

The decrease in riprap stability that results from the gravity component acting down the side slope is not the only mechanism determining the influence of the side slope angle on riprap stability. Also of importance is the influence of side slopes on the velocity and shear stress acting on the side slope. It is also required to determine the effect of side slopes on the placing of the maximum velocities adjacent to the toe of the side slopes. A third factor involves the interlocking of the riprap particles. On channel bottoms and mild side slopes the rock behaves more like individual units. On steep side slopes less than the angle of repose, the down slope gravity component causes greater interlocking forces. Also to be considered is the nature of riprap placed on a steep slope, where, if a local area of a revetment fails, riprap placed upslope will launch into the failed area.

To determine how the above mechanisms combine to define stability, tests of side slope riprap were conducted in a physical model bend. Only the outer bank side slope varied from test to test for each type of revetment structures. Side slopes of 1V:2H,

and 1V:3H were tested with model revetment having cubical ($d = 10$ mm) and hexagonal ($d = 5$ mm) type revetment structure and yielded the coefficients for Equation 5.13 as shown in Table 5.2-Table 5.5. The characteristic depth and velocity used in this design procedure for side slope revetment are the depth and depth-averaged velocity over the toe of the side slope for each type of revetment structure. The depth-averaged velocity over the toe in a channel bend can be determined using the measured local velocities at different depth over the toe of the revetment structure.

The physical model results shown in Table 5.2-Table 5.5 demonstrate the limited influence of side slope angle on riprap size, which is consistent with the findings in Hughes et al. (1983); Blodgett and McConaughy (1986), Ulrich (1987), and Maynard (1988). The last two investigations found that the Lane (1953) relation, which was used in many riprap design techniques, overestimates the influence of side slope angle when an angle of repose of 40° is used in the Lane equation. The side slope stability coefficients shown in Table 5.2-Table 5.5 cannot be compared to the Lane (1953) relation because the effects of the side slope velocity distribution, as well as the downslope gravity influence, is included in the Table 5.1 coefficients. Since the 2H:1V side slope results shown in Table 5.2-Table 5.5 for both cubical and hexagonal blocks gave relatively higher coefficient value in Eq. 5.13 than 3H:1V side slope therefore it can be concluded that flatter side slopes gives more stability for both the cases compared to higher side slope which is close to the angle of repose of the material. On the other hand, comparison between cubical and hexagonal type of revetments on same side slope gives better stability for hexagonal type revetment blocks. In addition, the model revetment structure tended to slide and roll down the side slope while being placed at higher slope than 2H:1V slope. Because of the placement problems, test run with higher slopes were not conducted in this study. Similarly, much flatter slope were not used in this study because of rare use of much flatter slopes than 3H:1V in Bangladesh. Therefore, this study recommends the use of side slopes of 2H:1V and 3H:1V for both the types of revetments.

Table 5.2 Stability Coefficients for Side Slope Revetment in Channel Bends at
velocity 50 cm/sec

Revetment Block Type	Side slope	C^*
Cubical Type Blocks	2H:1V	0.291
	3H:1V	0.29
Hexagonal Type Blocks	2H:1V	0.289
	3H:1V	0.286

Table 5.3 Stability Coefficients for Side Slope Revetment in Channel Bends at
velocity 60 cm/sec

Revetment Block Type	Side slope	C^*
Cubical Type Blocks	2H:1V	0.204
	3H:1V	0.2
Hexagonal Type Blocks	2H:1V	0.198
	3H:1V	0.197

Table 5.4 Stability Coefficients for Side Slope Revetment in Channel Bends at
velocity 70 cm/sec

Revetment Block Type	Side slope	C^*
Cubical Type Blocks	2H:1V	0.148
	3H:1V	0.145
Hexagonal Type Blocks	2H:1V	0.144
	3H:1V	0.142

Table 5.5 Stability Coefficients for Side Slope Revetment in Channel Bends at
velocity 80 cm/sec

Revetment Block Type	Side slope	C^*
Cubical Type Blocks	2H:1V	0.114
	3H:1V	0.111
Hexagonal Type Blocks	2H:1V	0.110
	3H:1V	0.108

The stability of riprap bank revetment is affected by the steepness of channel side slopes. Side slopes should not be steeper than 1V on 2H, except in special cases where it may be economical to use larger hand-placed stone keyed well into the bank. Side slopes from 1V on 2H to 1V on 3H are recommended for riprap stability. The size of stone required resisting the erosive forces of channel flow increases when the side slope angle approaches the angle of repose of a riprap revetment. Rapid water-level recession and piping-initiated failures are other factors in determining channel side slopes.

From the values of stability coefficients C^* for different side slope for different revetments in channel bends presented in Table 5.2 to Table 5.5 it is seen that for all the velocity ranges of 50 – 80 cm/sec for the same slope Hexagonal showed lower C^* value. This means Hexagonal blocks remains more stable than cubical blocks in the same velocity ranges and same side slope. Again comparing same type of revetment in same velocity ranges in different side slopes, it is observed that blocks remain more stable in 3:1 slope than 2:1 slope. These trends can be also seen in the plots of stability coefficient C^* vs. Froude Number in Figure 5.1 to Figure 5.4 for cubical and hexagonal blocks in 2:1 and 3:1 slopes

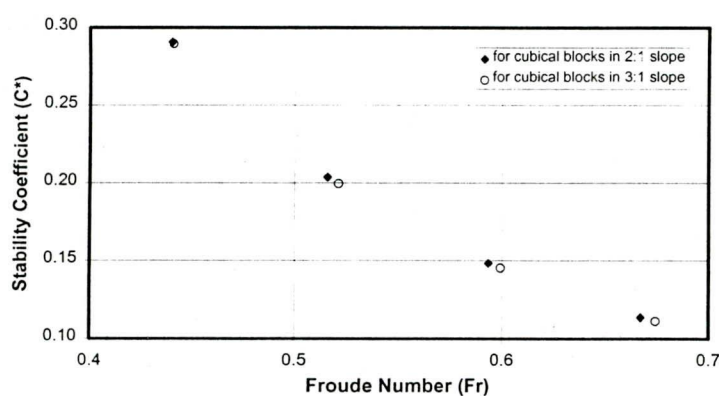


Figure 5.1 Stability coefficient C^* vs. Froude number for cubical blocks in 2:1 and 3:1 slope

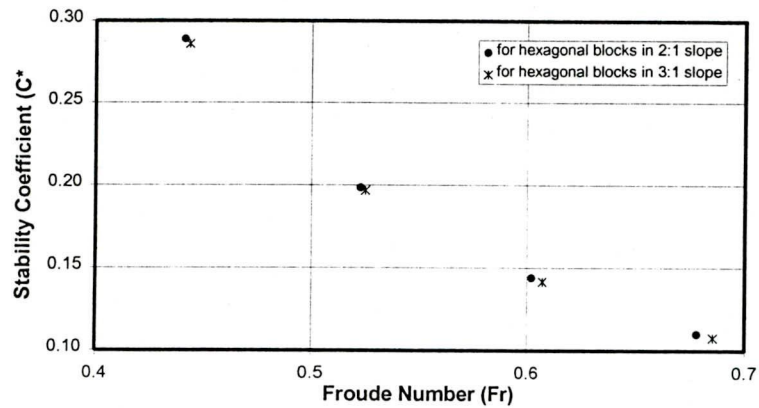


Figure 5.2 Stability coefficient C^* vs. Froude number for hexagonal blocks in 2:1 and 3:1 slope

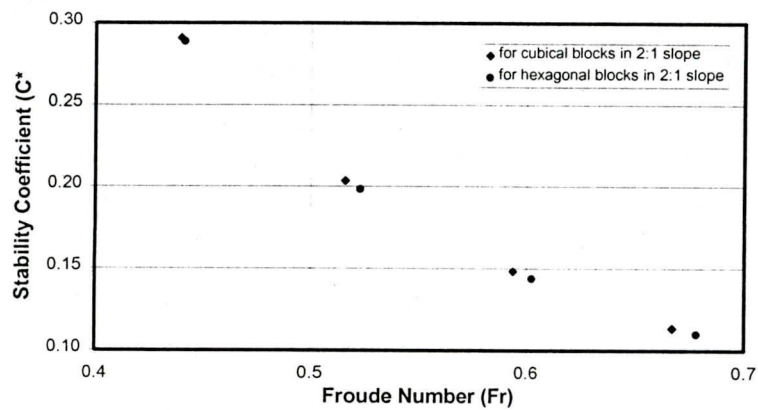


Figure 5.3 Stability coefficient C^* vs. Froude number for cubical and hexagonal blocks in 2:1 slope

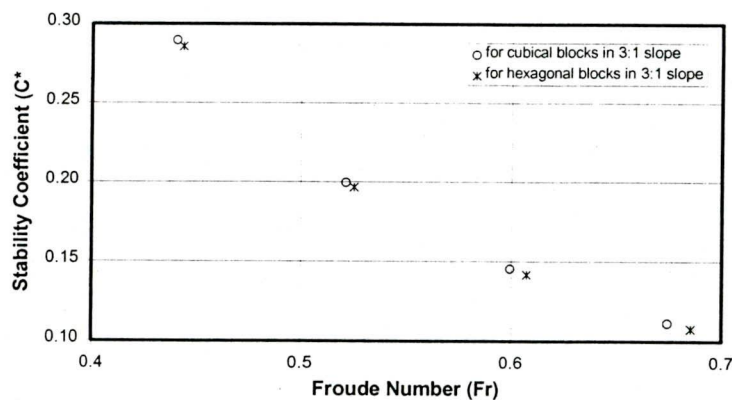


Figure 5.4 Stability coefficient C^* vs. Froude number for cubical and hexagonal blocks in 3:1 slope

5.2.3 Pattern of velocity plots

Velocities profile along the side slope of the structure at different sections of the protected area with different types of revetments in different scenario run are presented in Appendix D.

5.2.4 Measurement of Bank Profile along the Revetment Structure

The developments of bank configuration protected by revetment with increasing flow velocities were established by analysing the data on computer. For different sections A to H, the plotted results of the development of bank configuration are shown in Figure 5.5 to Figure 5.12. The nature of local scouring/erosion from section A to H by increasing flow velocities can be seen from these figures for two different shape of revetment at different bank slopes. These figures give a qualitative idea about the damage of the different types of revetment structures at different flow velocities.

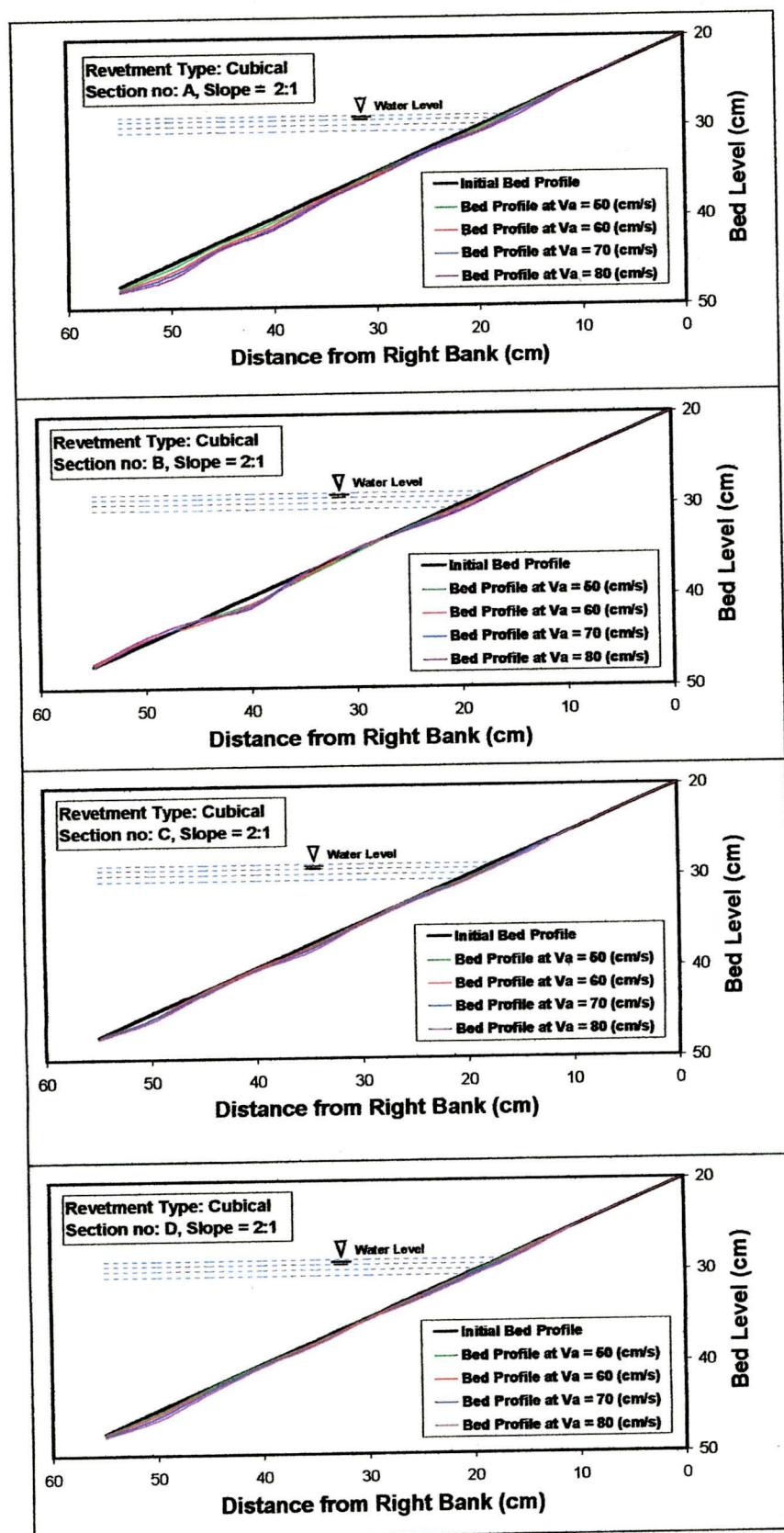


Fig. 5.5 Bed profile of bank at different sections A to D protected by cubical revetment with 2:1 bank slope at increasing flow velocities

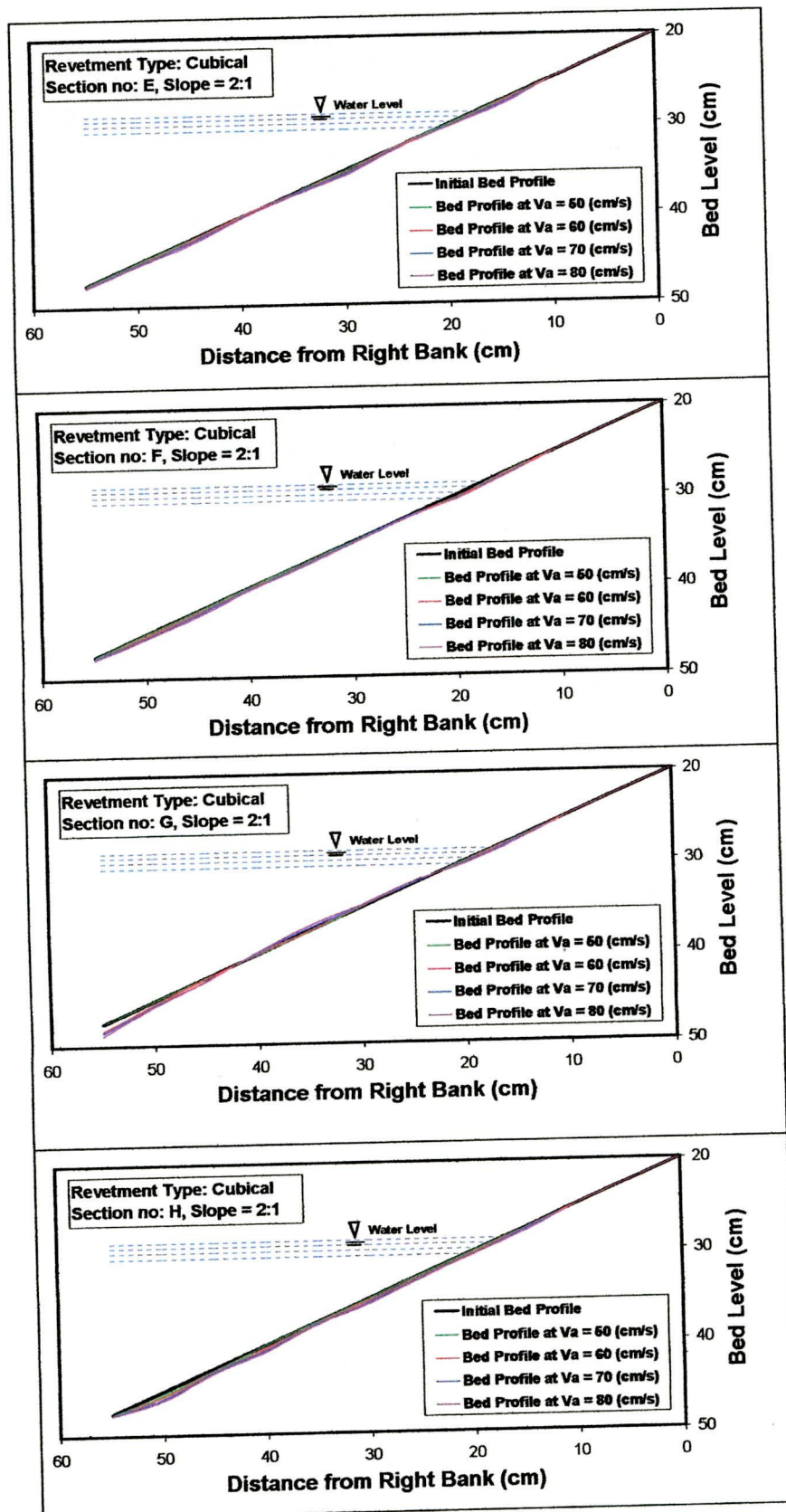


Fig. 5.6 Bed profile of bank at different sections E to H protected by cubical revetment with 2:1 bank slope at increasing flow velocities

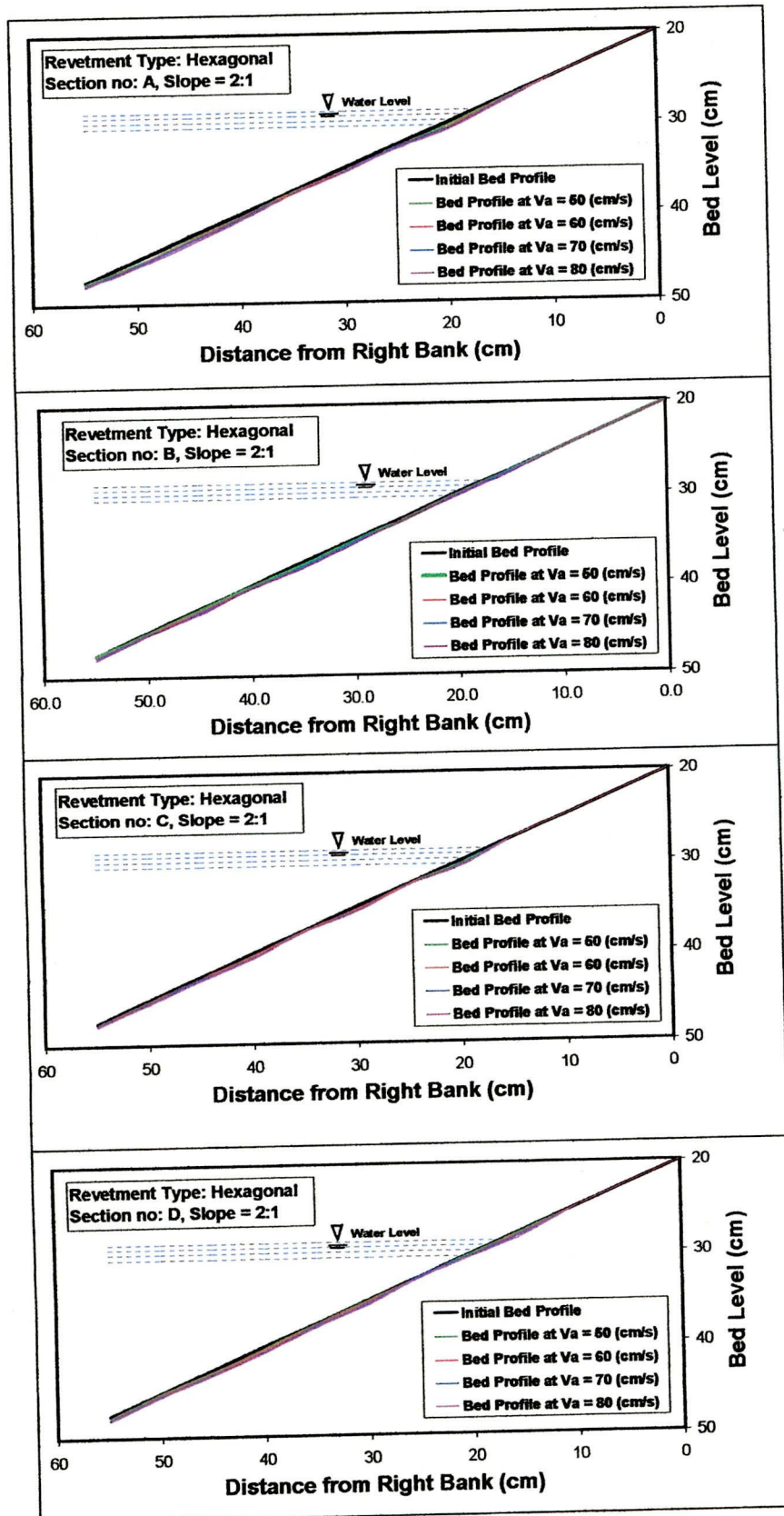


Fig. 5.7 Bed profile of bank at different sections A to D protected by hexagonal revetment with 2:1 bank slope at increasing flow velocities

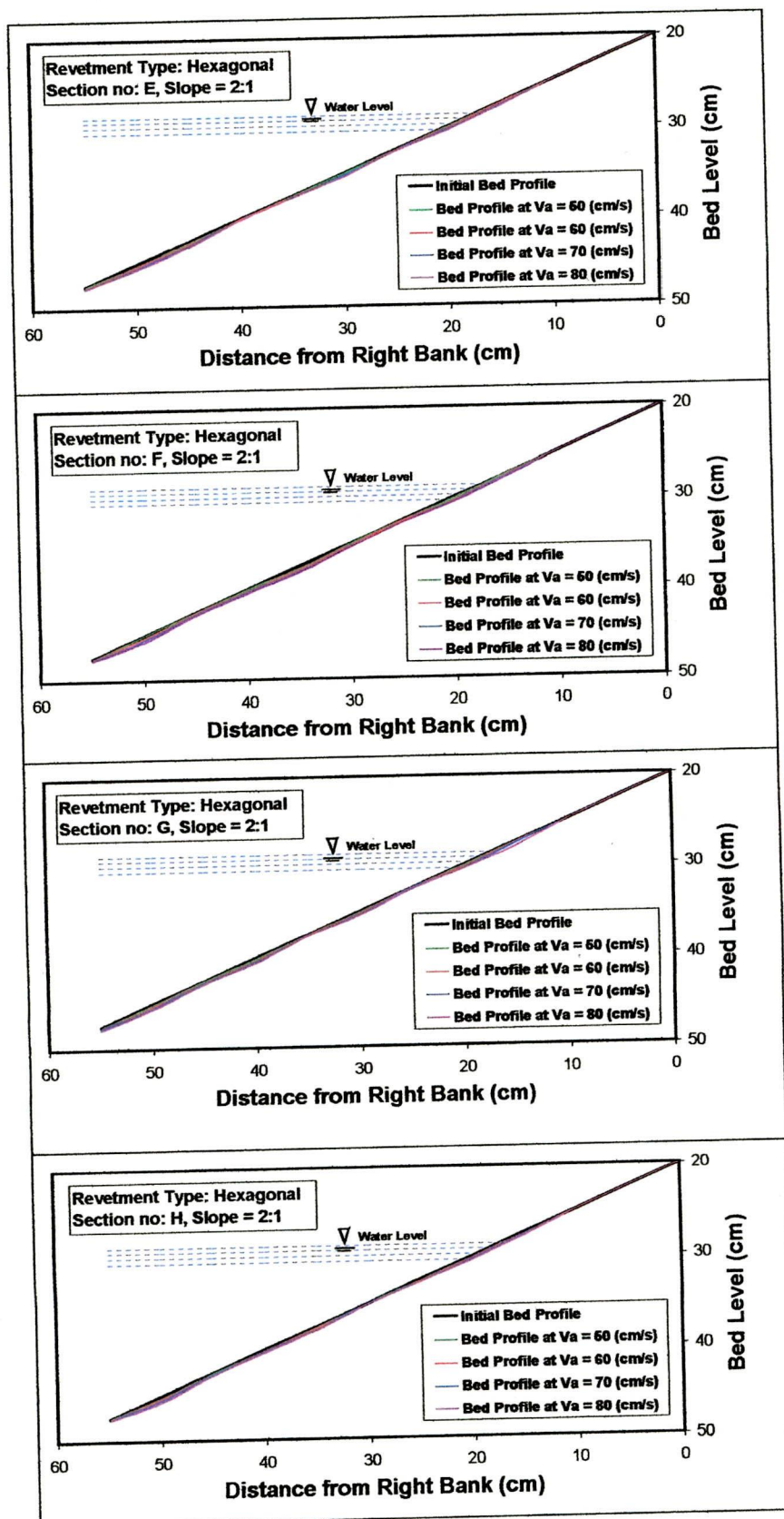


Fig. 5.8 Bed profile of bank at different sections E to H protected by hexagonal revetment with 2:1 bank slope at increasing flow velocities

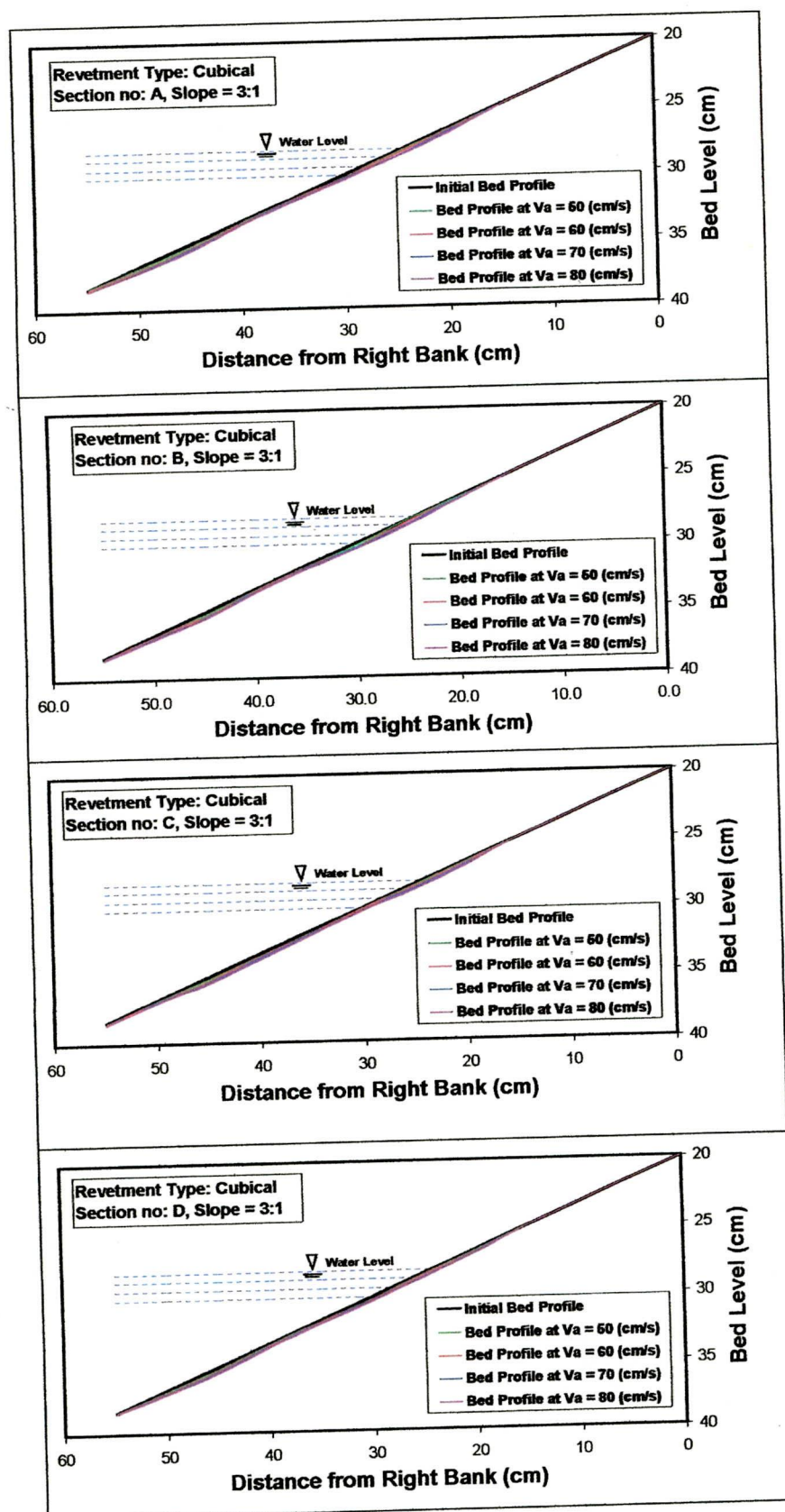


Fig. 5.9 Bed profile of bank at different sections A to D protected by cubical revetment with 3:1 bank slope at increasing flow velocities

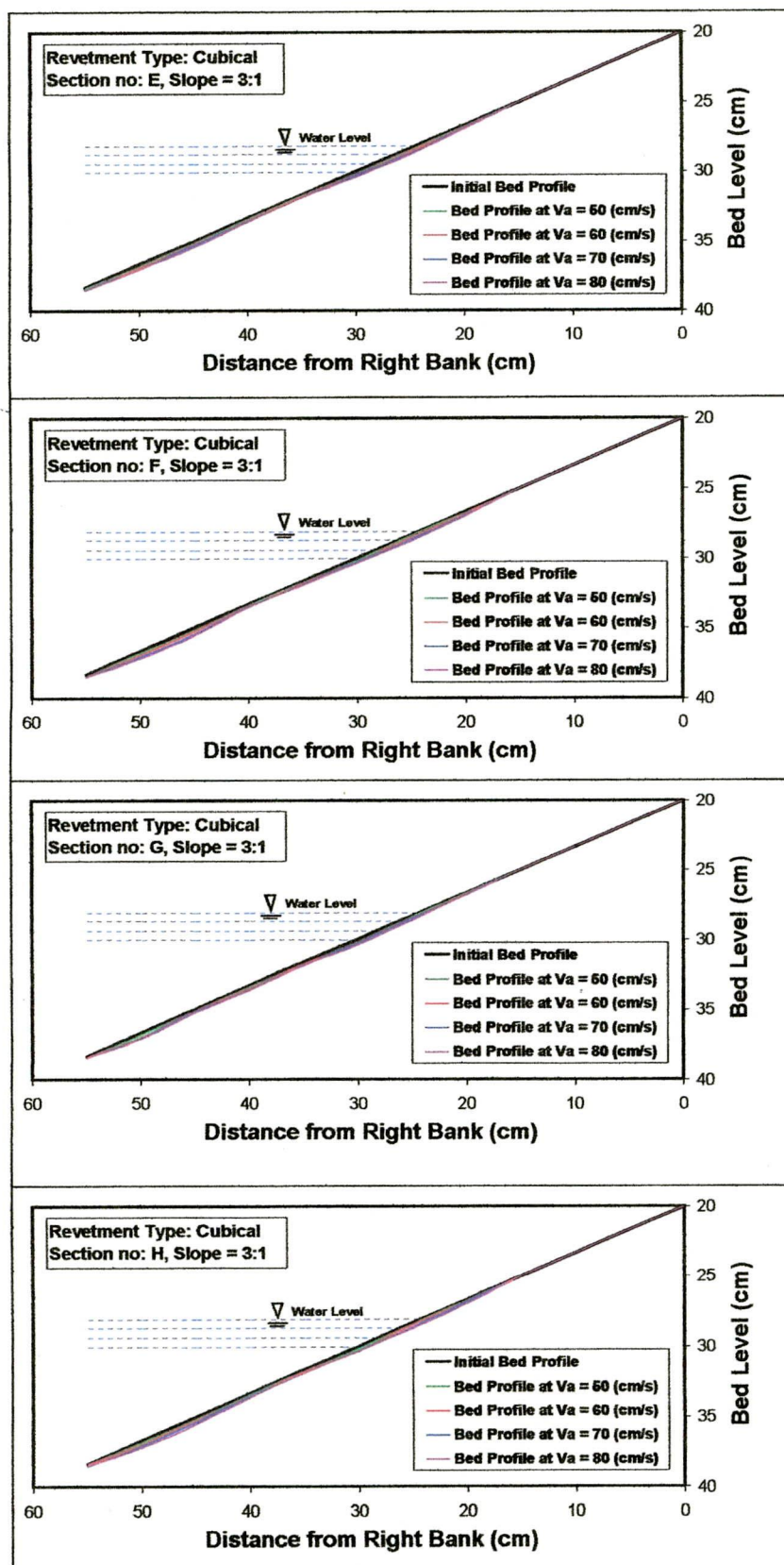


Fig. 5.10 Bed profile of bank at different sections E to H protected by cubical revetment with 3:1 bank slope at increasing flow velocities

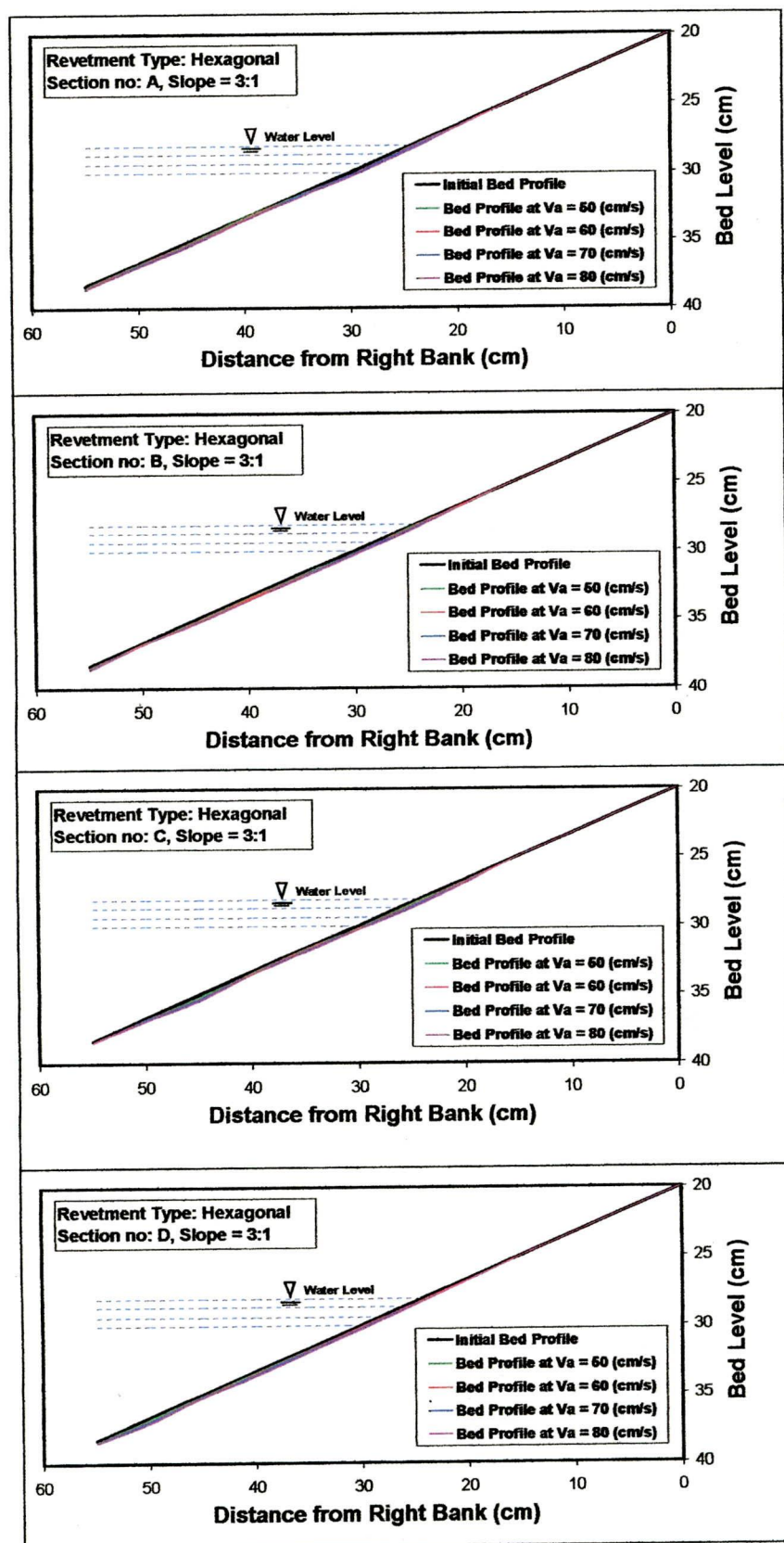


Fig. 5.11 Bed profile of bank at different sections A to D protected by hexagonal revetment with 3:1 bank slope at increasing flow velocities

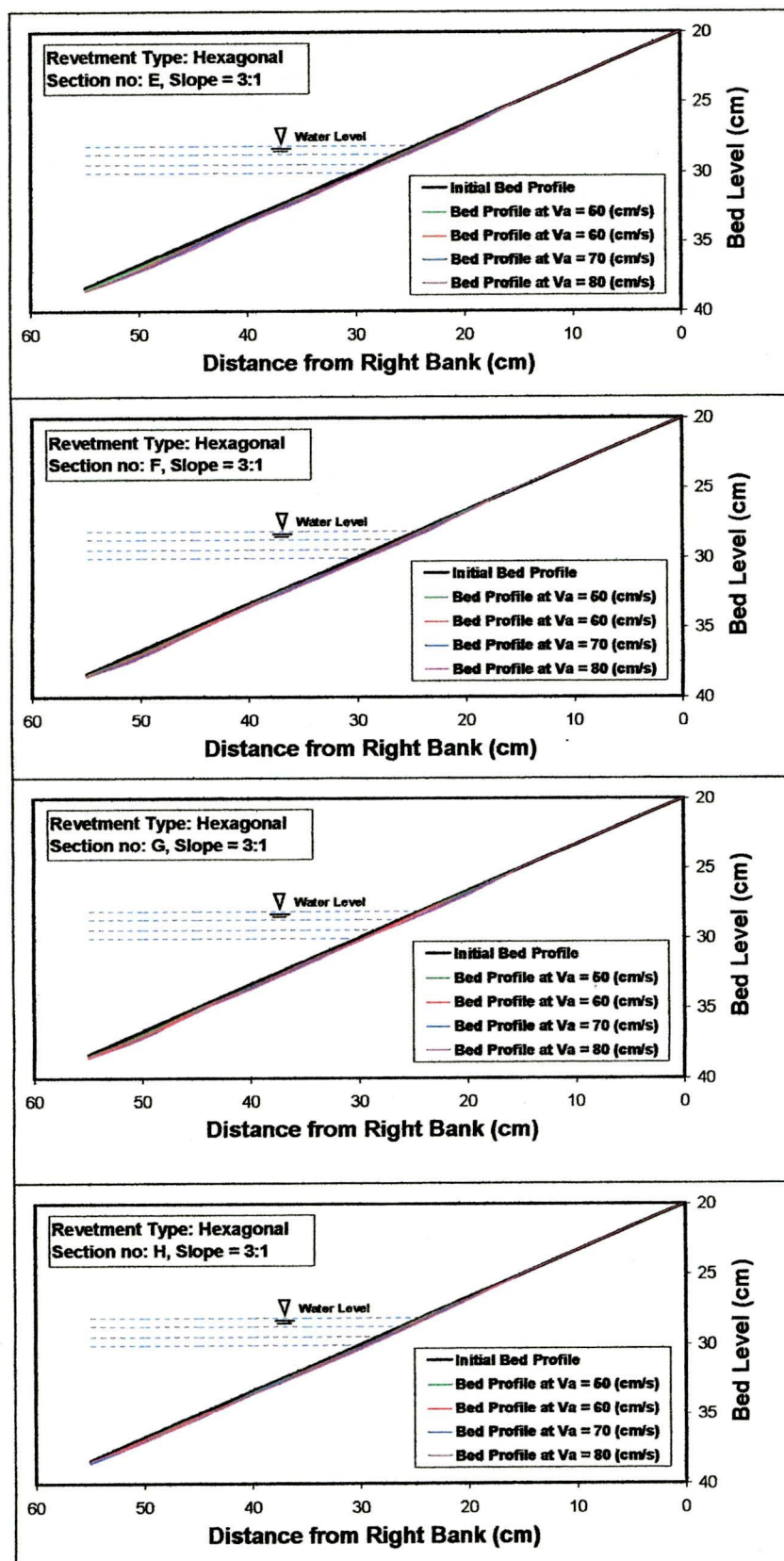


Fig. 5.12 Bed profile of bank at different sections E to H protected by hexagonal revetment with 3:1 bank slope at increasing flow velocities

5.2.4.1 Analysis of Bank Profile Data

Damage Level

The damage to the armour layer can be given as a percentage of displaced rocks related to a certain area (the whole or a part of the layer). In this case, however, it is difficult to compare various structures as the damage figures are related to different totals for each structure. Another possibility is to describe the damage by the erosion area around still-water level. When this erosion area is related to the size of the rocks, a dimensionless damage level is presented which is independent of the size (slope angle and height) of the structure. This damage level is defined by (Pilarczyk, 1998):

$$S = \frac{A_e}{D_{n50}^2}$$

where:

S = damage level

A_e = erosion area around still-water level

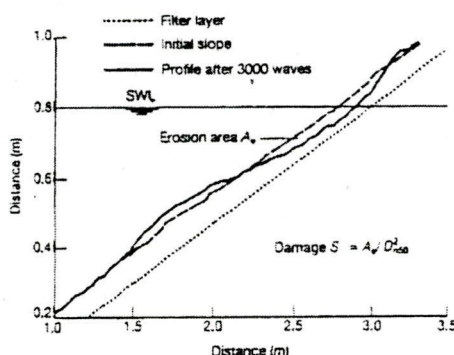


Figure 5.13 Damage S based on erosion area A_e (Source: Pilarczyk, 1998)

A plot of a structure with damage is shown in Figure 5.13, the damage level taking both settlement and displacement into account. A physical description of the damage, S , is the number of squares with a side D_{n50} that fit into the erosion area. Another description of S is the number of cubic stones with a side of D_{n50} eroded within a D_{n50} -wide strip of the structure. The actual number of stones eroded within this strip can be more or less than S , depending on the porosity, the grading of the armour rocks and the shape of the rocks. Generally the actual number of rocks eroded in a D_{n50} -

wide strip is equal to 0.7 to 1 times the damage S . The limits of S mainly depend on the slope angle of the structure.

The effects of armour shape on stability have been described by the following figures. Two classes of revetment with different shape classification such as cubical and hexagonal are tested in this experiment. As expected, hexagonal shaped revetment structure suffered less damage than the cubical shapes for both the bank slope conditions. Similarly, the performances of each of the revetment types with different bank slopes were also tested. Here, the mild sloped (i.e. 3:1) revetment structures (both cubical and hexagonal) exhibited higher stability than the steeper sloped one.

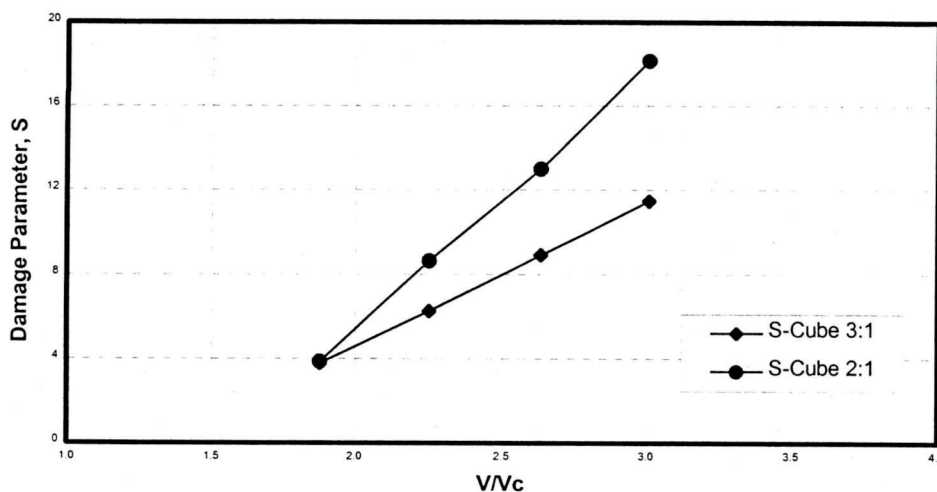


Figure 5.14 Comparison of Damage Parameter and V/V_c for Cubical blocks in 2:1 and 3:1 slopes

Figure 5.14 shows that for cubical blocks damage parameter S increases in slower rate in case of 3:1 slope than 2:1 slope. This indicates that cubical blocks in 2:1 slope are less susceptible to damage. At high V/V_c ratio 2:1 slope S is 58% more than that of 3:1

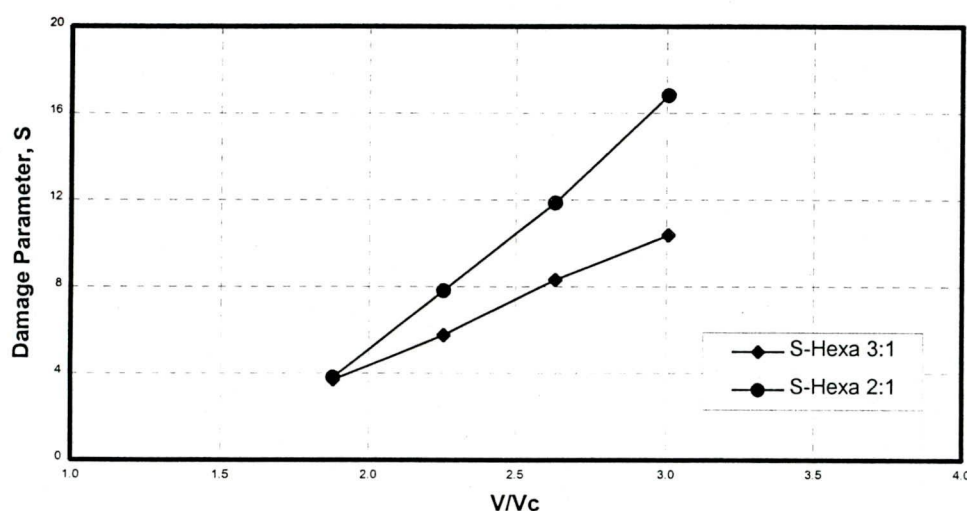


Figure 5.15 Comparison of Damage Parameter and V/V_c for Hexagonal blocks in 2:1 and 3:1 slopes

Figure 5.15 shows a similar trend like cubical blocks. Here also, in case of hexagonal blocks, damage parameter S increases in slower rate in case of 3:1 slope than that of 2:1 slope. At low V/V_c the damage parameter for 2:1 and 3:1 slope are almost same.

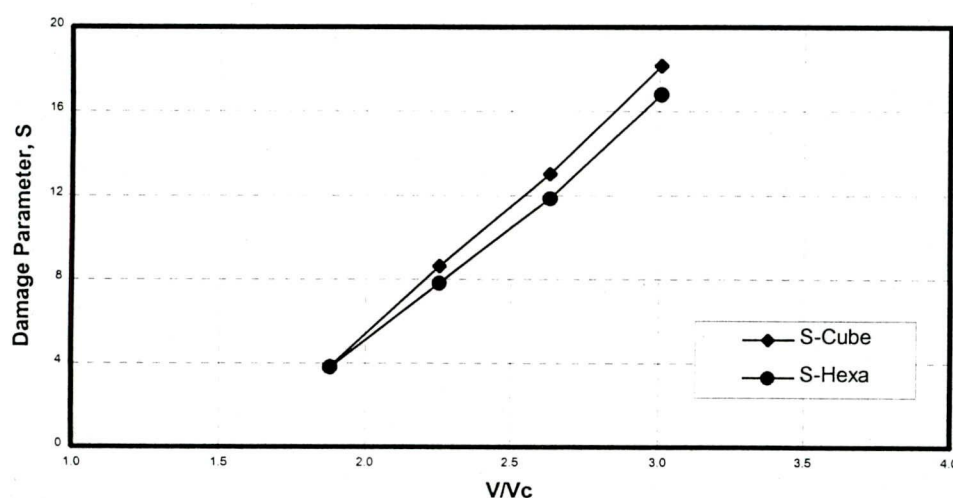


Figure 5.16 Comparison of Damage Parameter and V/V_c with Cubical blocks and Hexagonal blocks in 2:1 slope.

Analysing figure 5.16 it can be said that for same slope performance of both the cubical and hexagonal blocks are almost similar. But the damage parameter S is always lower for hexagonal blocks than cubical blocks.

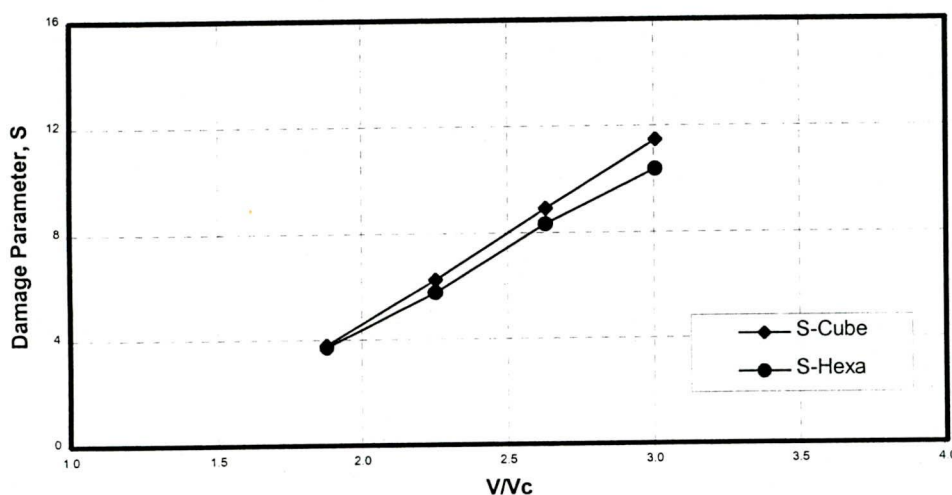


Figure 5.17 Comparison of Damage Parameter and V/V_c with Cubical blocks and Hexagonal blocks in 3:1 slope

From figure 5.17 it can be concluded that Damage parameter S for both cubical and hexagonal blocks in 3:1 slope are much lower than those at 2:1 slope. At high V/V_c ratio in case of 2:1 slope the value of damage parameter are higher than 16 whereas in case of 3:1 it is below 12. This figure also reveals that in same slope hexagonal blocks get less damaged than cubical blocks.

Contour Plots of Local Scour/Erosion Pattern

In addition, local scour/erosion pattern along the right bank of the protected study area has also been analysed at different velocities ranging from 50 cm/s to 80 cm/s. With the help of contour plot, these data were analysed and found that with the increase in the value of average channel velocities, aerial extent of local scour/erosion also increases. Fig. 5.18 to 5.21 shows these types of contour plots. These plots mainly gives a qualitative idea about the development of local scour/erosion with increasing rather than a quantitative view. Almost in all the cases, it is found that local scour/erosion initiates from the upstream end of the revetment structure and it propagates to the downstream end of the structure with the increasing flow velocities.

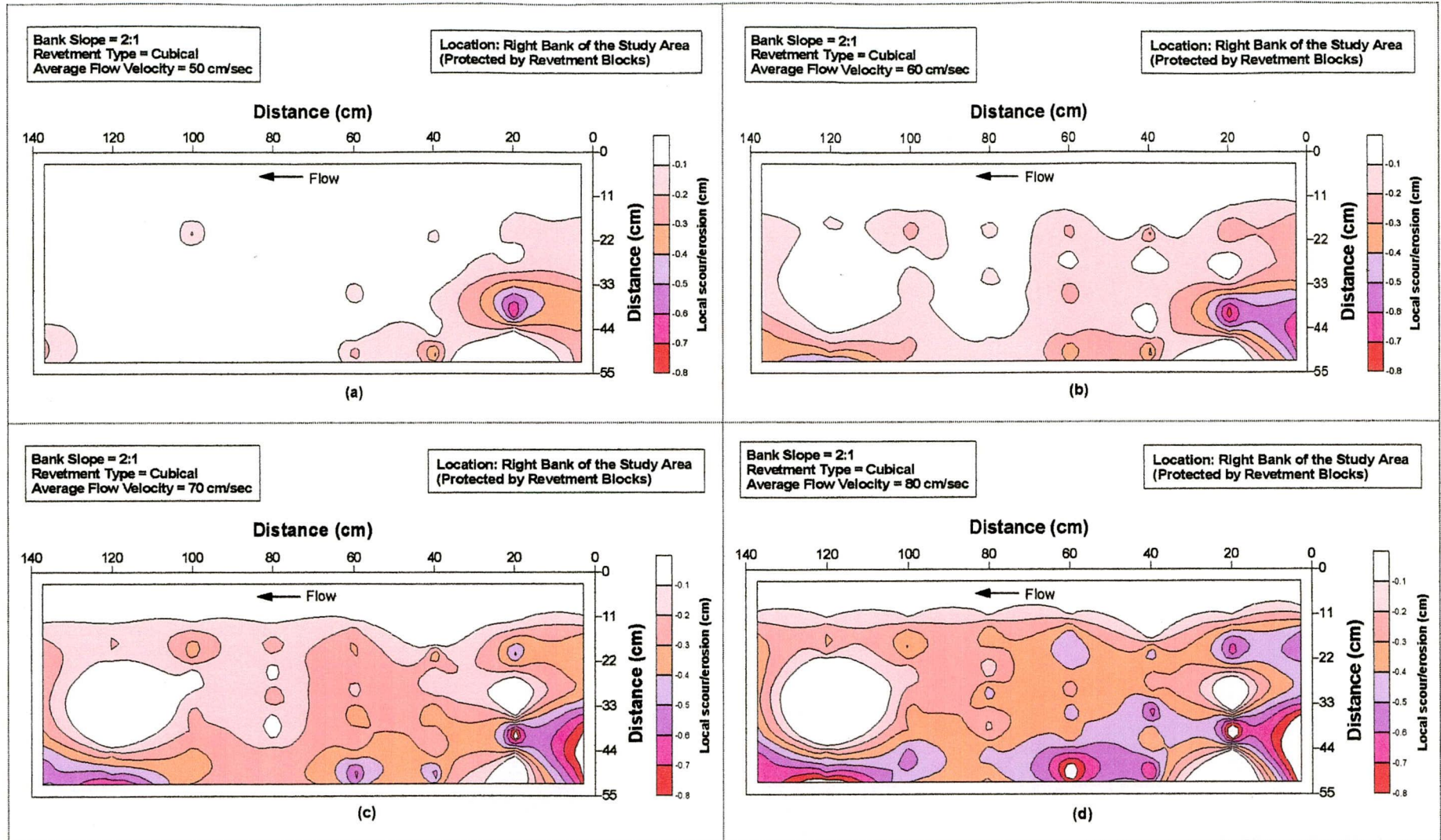


Fig. 5.18 Local scour/erosion pattern along the right bank of the protected study area by cubical revetment blocks with 2:1 bank slope at different average flow velocities: (a) $V_{avg} = 50$ cm/sec, (b) $V_{avg} = 60$ cm/sec, (c) $V_{avg} = 70$ cm/sec, (d) $V_{avg} = 80$ cm/sec

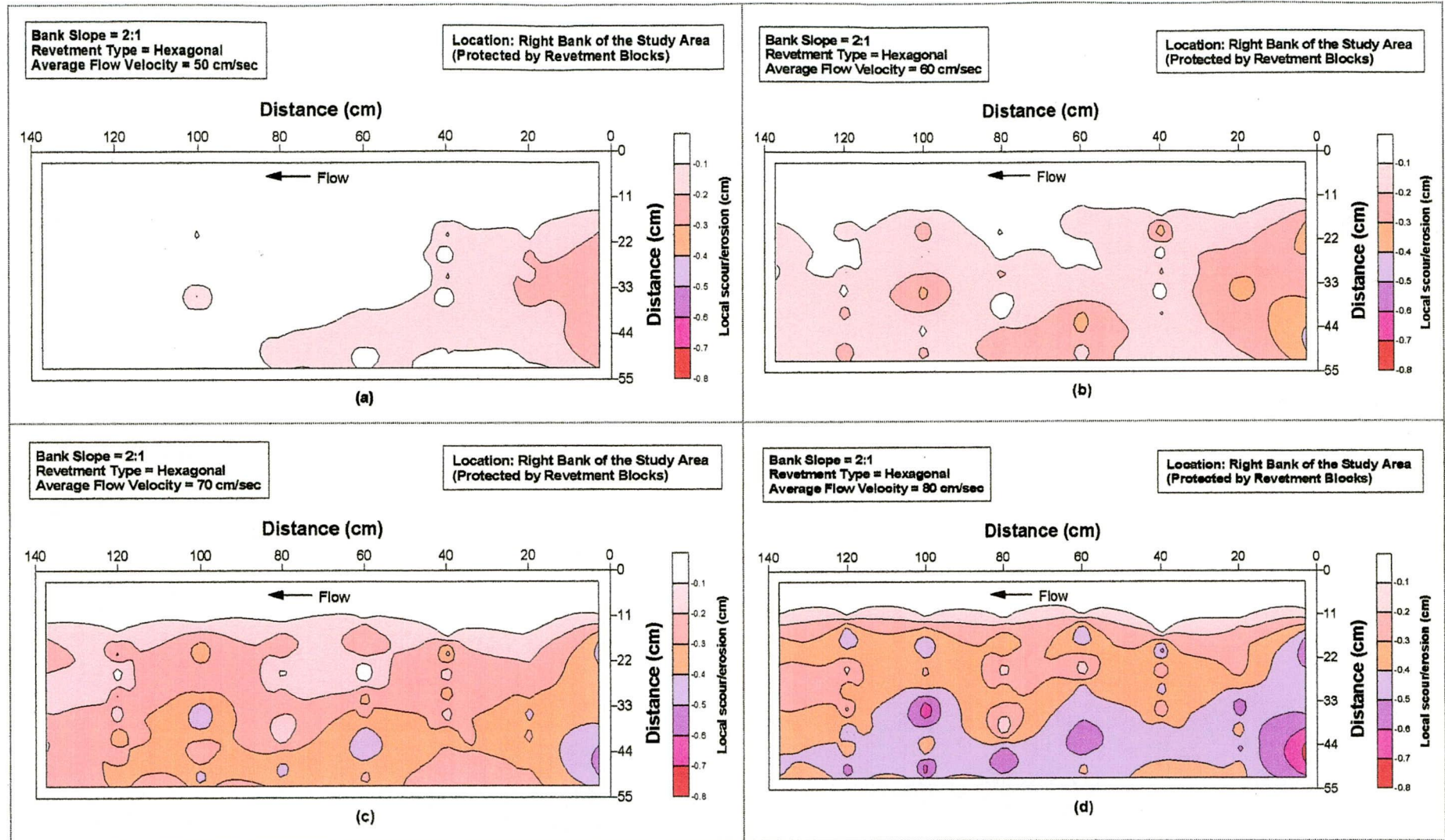


Fig. 5.19 Local scour/erosion pattern along the right bank of the protected study area by hexagonal revetment blocks with 2:1 bank slope at different average flow velocities: (a) $V_{avg} = 50$ cm/sec, (b) $V_{avg} = 60$ cm/sec, (c) $V_{avg} = 70$ cm/sec, (d) $V_{avg} = 80$ cm/sec

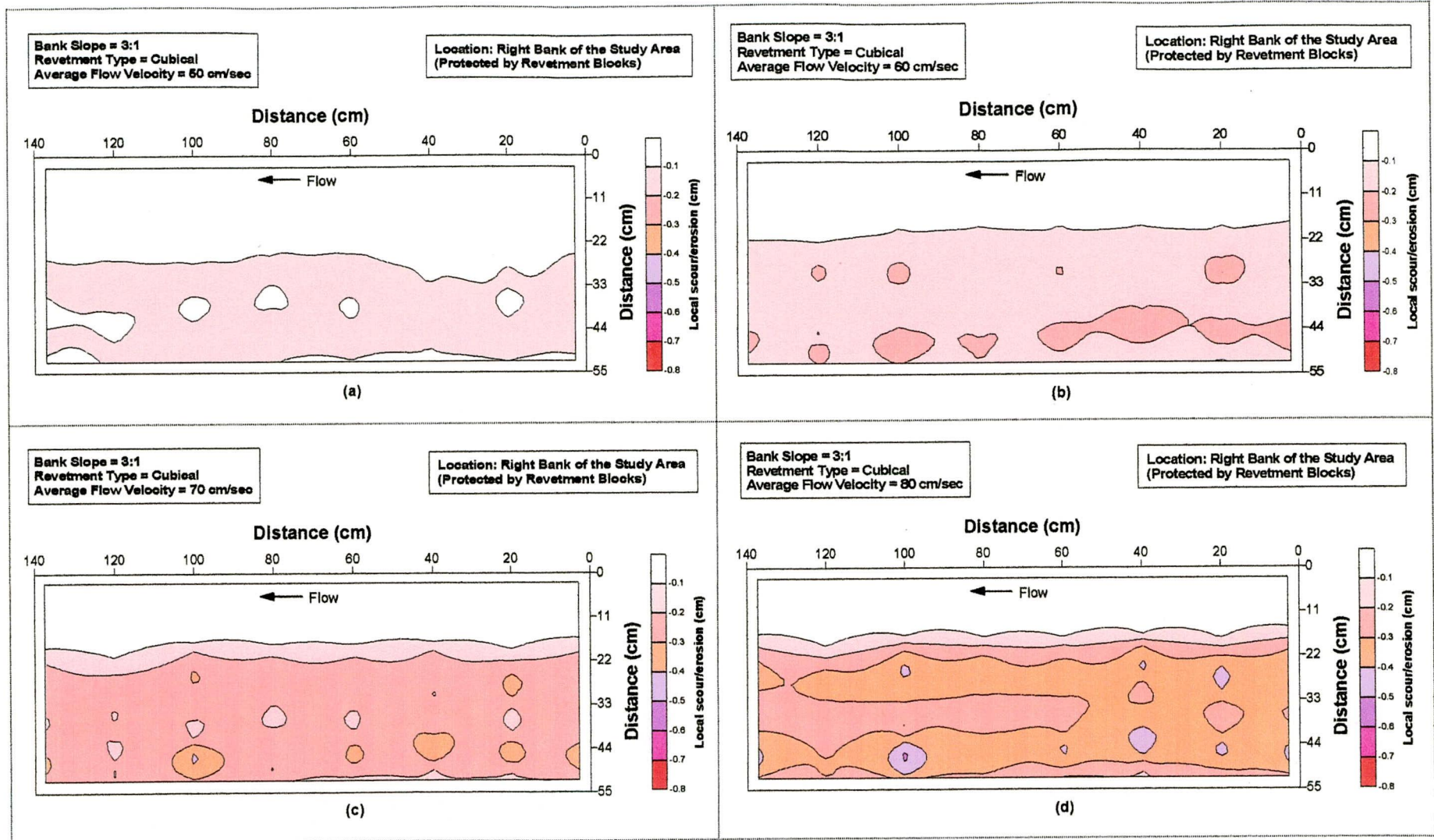


Fig. 5.20 Local scour/erosion pattern along the right bank of the protected study area by cubical revetment blocks with 3:1 bank slope at different average flow velocities: (a) $V_{avg} = 50$ cm/sec, (b) $V_{avg} = 60$ cm/sec, (c) $V_{avg} = 70$ cm/sec, (d) $V_{avg} = 80$ cm/sec

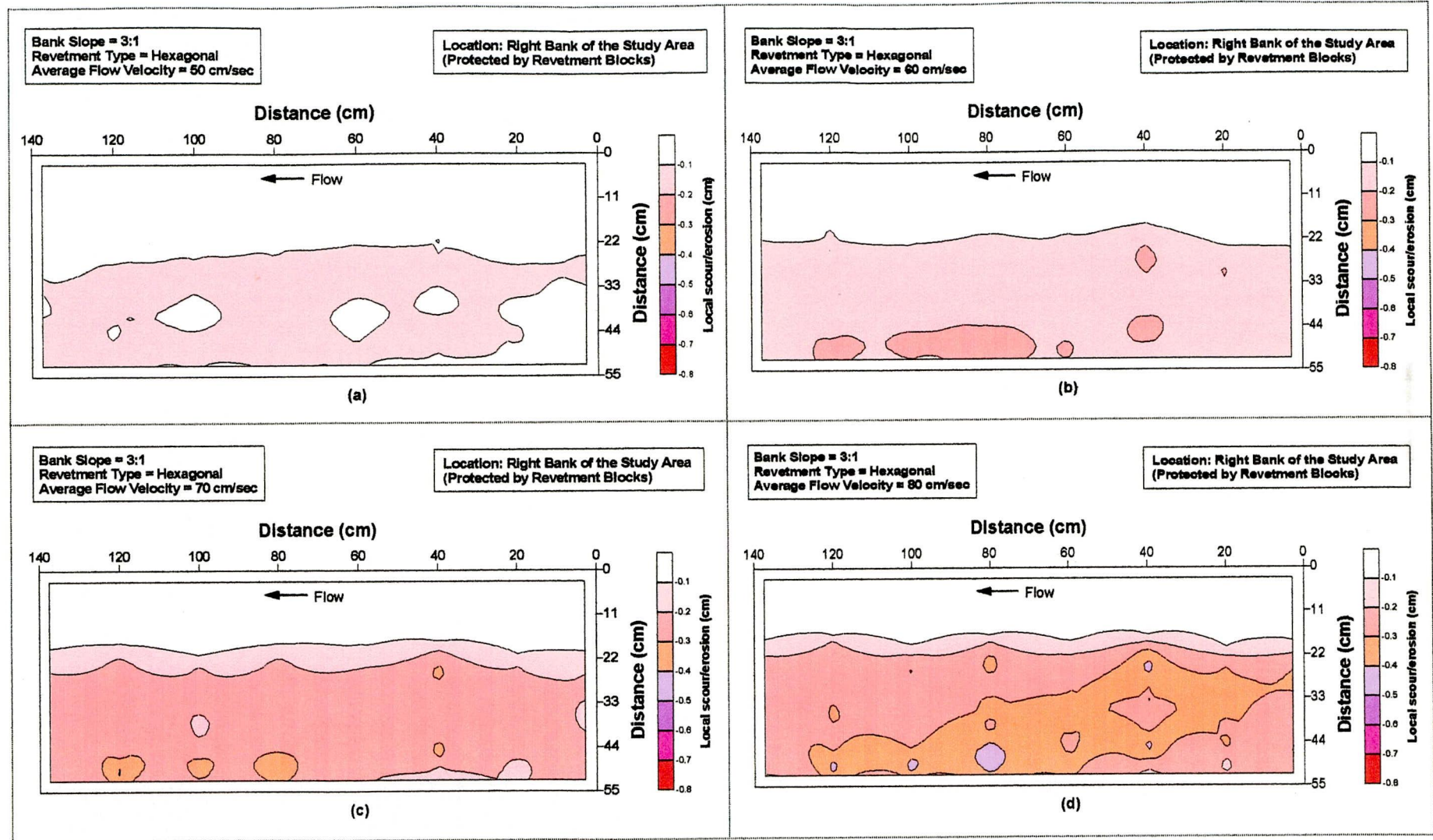


Fig. 5.21 Local scour/erosion pattern along the right bank of the protected study area by hexagonal revetment blocks with 3:1 bank slope at different average flow velocities: (a) $V_{avg} = 50$ cm/sec, (b) $V_{avg} = 60$ cm/sec, (c) $V_{avg} = 70$ cm/sec, (d) $V_{avg} = 80$ cm/sec

CHAPTER SIX

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 *Introduction*

The physical model facilities that has been developed for short term and long term research activities on alluvial channel was used to study the performance of cubical block and hexagonal block as river bank protection measure under different velocity ranges and bank slope conditions.

6.2 *Conclusions*

The following conclusions are drawn from this study:

1. For all the velocity ranges of 50 – 80 cm/sec, the hexagonal blocks remained more stable than cubical blocks in the same velocity ranges and same side slope.
2. Analyzing local scour pattern along the protected area it was found that in almost all the cases, local scour initiates from the upstream end of the revetment structure and propagates to the downstream end of the structure with the increasing flow velocities.
3. Local depth-averaged velocity that can be used for the design of revetment structure and stability analysis in a bend is found to be 14.5-18% higher than the average velocity of the channel.
4. With increasing V/V_c ratio damage parameter S increases for both 2H:1V and 3H:1V slopes with hexagonal and cubical blocks.
5. With increasing V/V_c ratio the rate of increase of damage parameter S is less for 3H:1V slope than 2H:1V slope for both cubical and hexagonal blocks. This means blocks in 3H:1V slope is less susceptible to damage than those at 2H:1V slope.
6. In same slope, with increasing V/V_c ratio, damage parameter S of the hexagonal blocks is always less than cubical blocks. From this it can be concluded that in the same side slope condition and same velocity range the performance of hexagonal block is better than cubical blocks.

7. At slope higher than 2H:1V both the hexagonal and cubical blocks tend to slide or roll down the side slope while being placed. Again slopes flatter than 3H:1V is of rare use in Bangladesh. So, this study recommend the use of side slope of 2H:1V and 3H:1V for both the types of revetments.
8. From the study on bank protection by revetment it was found that hexagonal blocks required 50% less amount of material as compared to cubical blocks of equivalent weight to cover the selected bank protection area.
9. Time required for placement of two kinds of blocks show that cubical blocks require 40% more time than as compared to hexagonal block due to their smaller shape and size.

6.3 *Recommendations for Future Study*

The following recommendations can be drawn:

- (i) More powerful new pump may be used to get more or less constant discharge through the model run.
- (ii) Finally adopted design of sediment tank may be re-checked to ensure proper sedimentation in the sediment trap.
- (iii) Addition of sediment feeding system may be done.
- (iv) A shed over the physical model facility may be provided so that it can be used through out the year.
- (v) Provision of enough light may be arranged to work in model at night or at less day light.
- (vi) Guide line given here to design the Hexagonal blocks may be taken as initial one. Before applying these blocks in the actual field further experiments should be done using different size of hexagonal blocks on different bank slope and flow condition to gain a thorough knowledge of its behaviour.

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APPENDICES

APPENDIX A

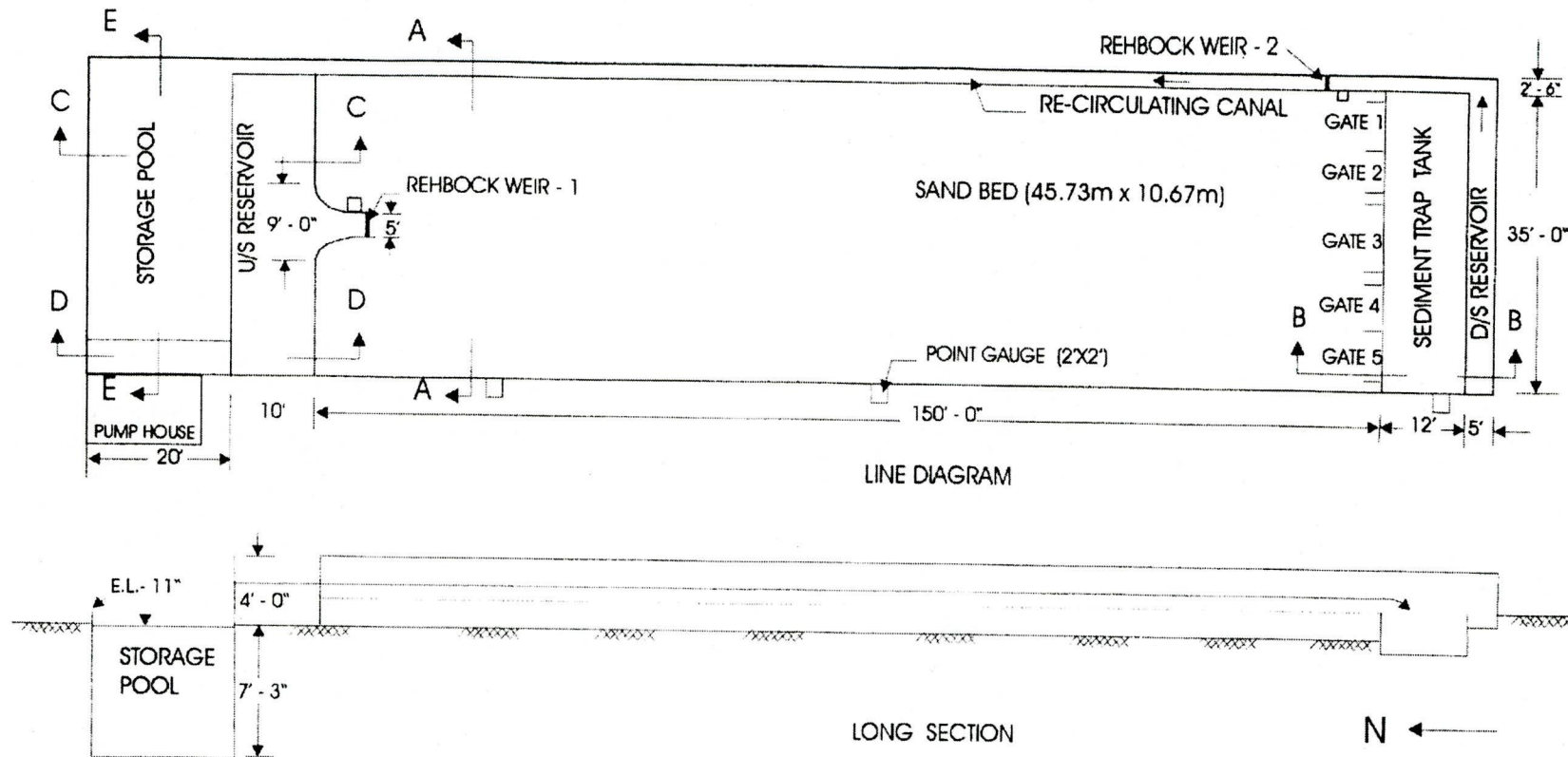


Figure A.1 Final plan of the physical model facility (Source: Sarkar S.M.R, 2001)

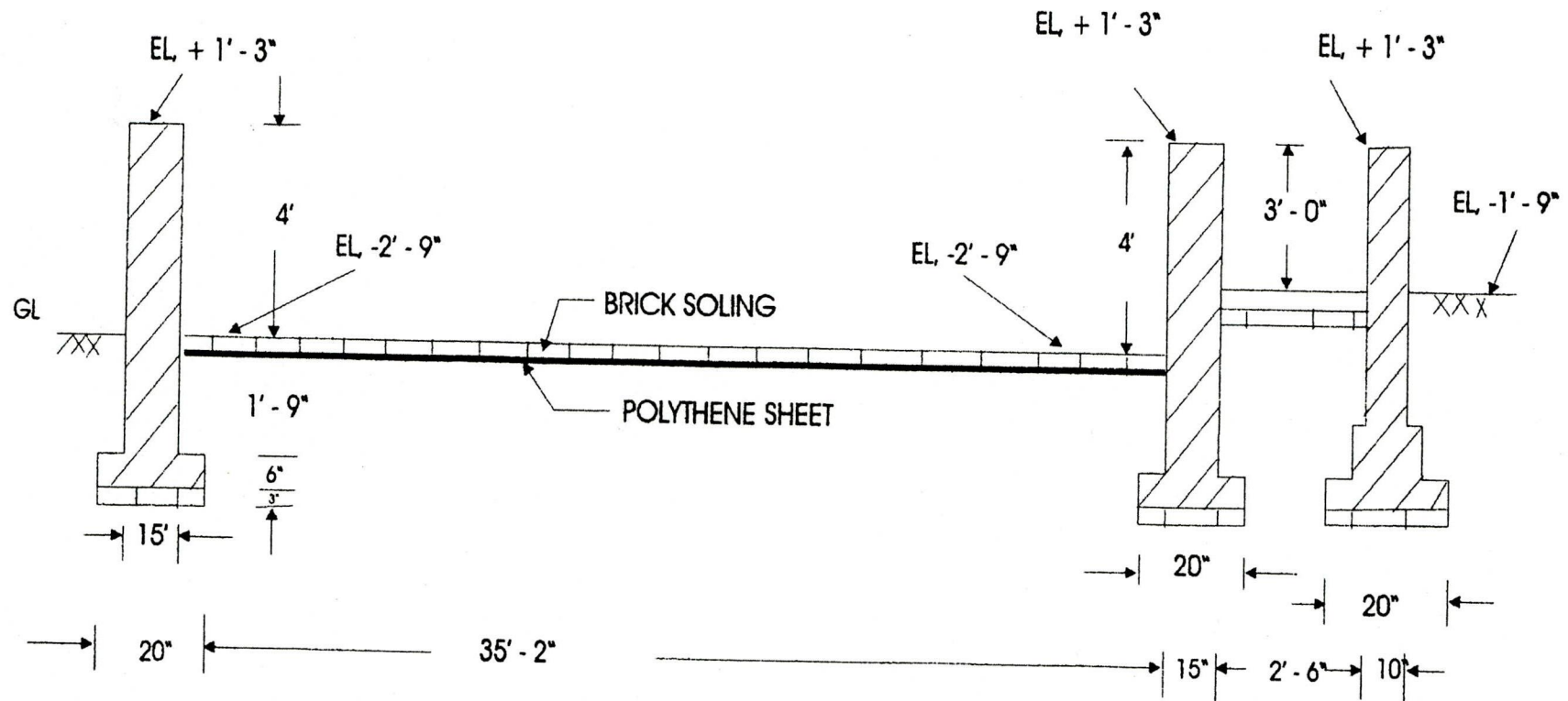


Figure A.2 Section A-A of the final plan of the physical model facility (Source: Sarkar S.M.R, 2001)

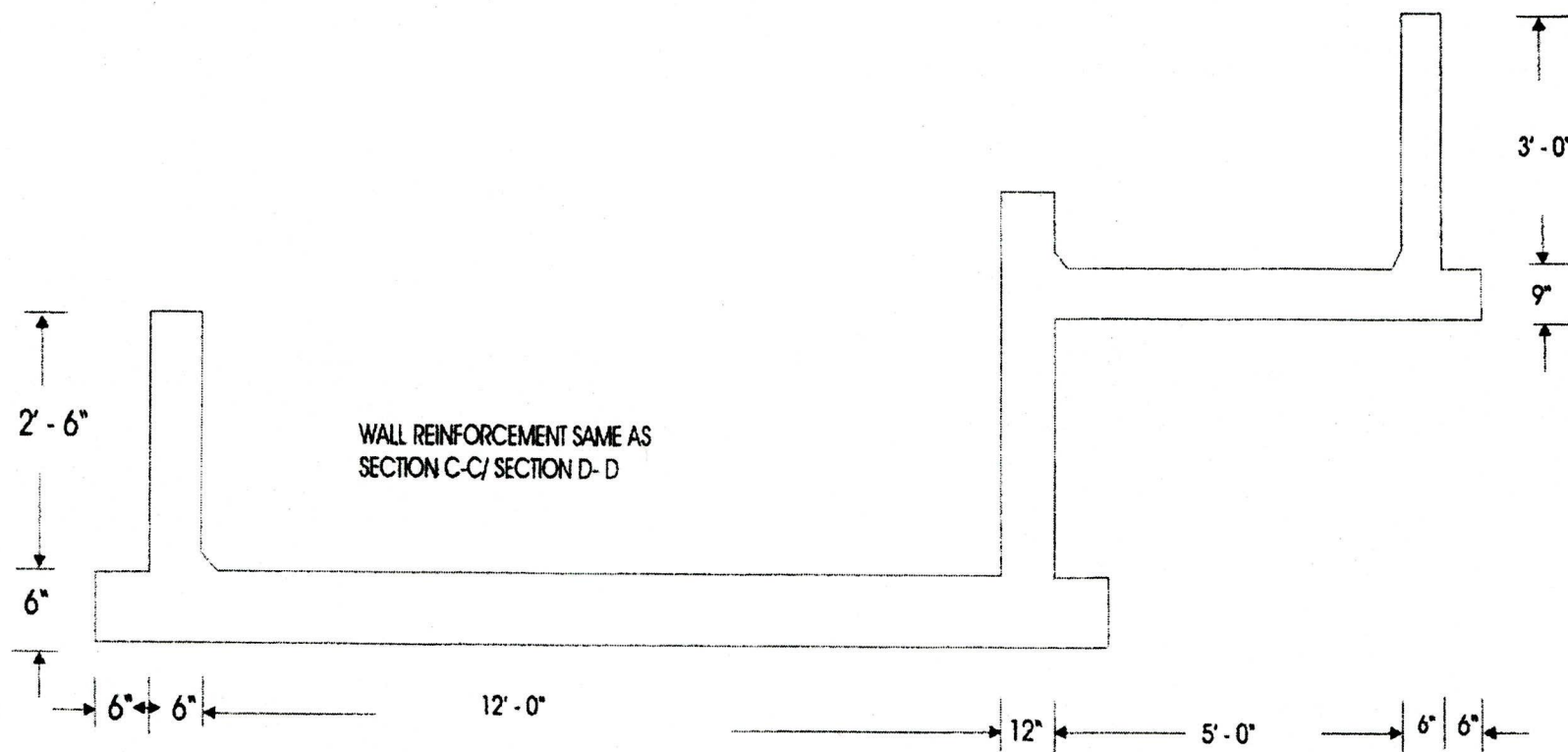


Figure A.3 Section B-B of the final plan of the physical model facility (Source: Sarkar S.M.R, 2001)

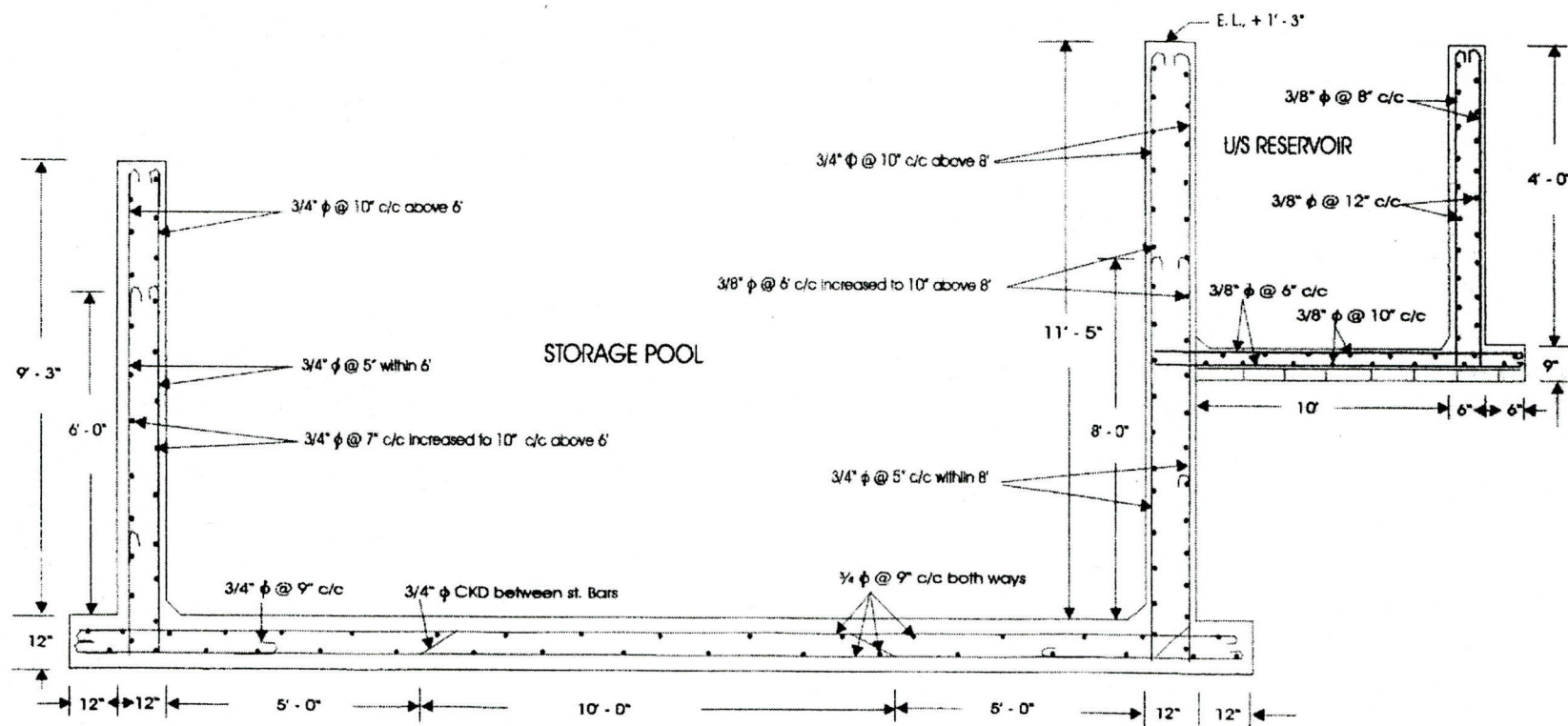


Figure A.5 Section D-D of the final plan of the physical model facility (Source: Sarkar S.M.R, 2001)

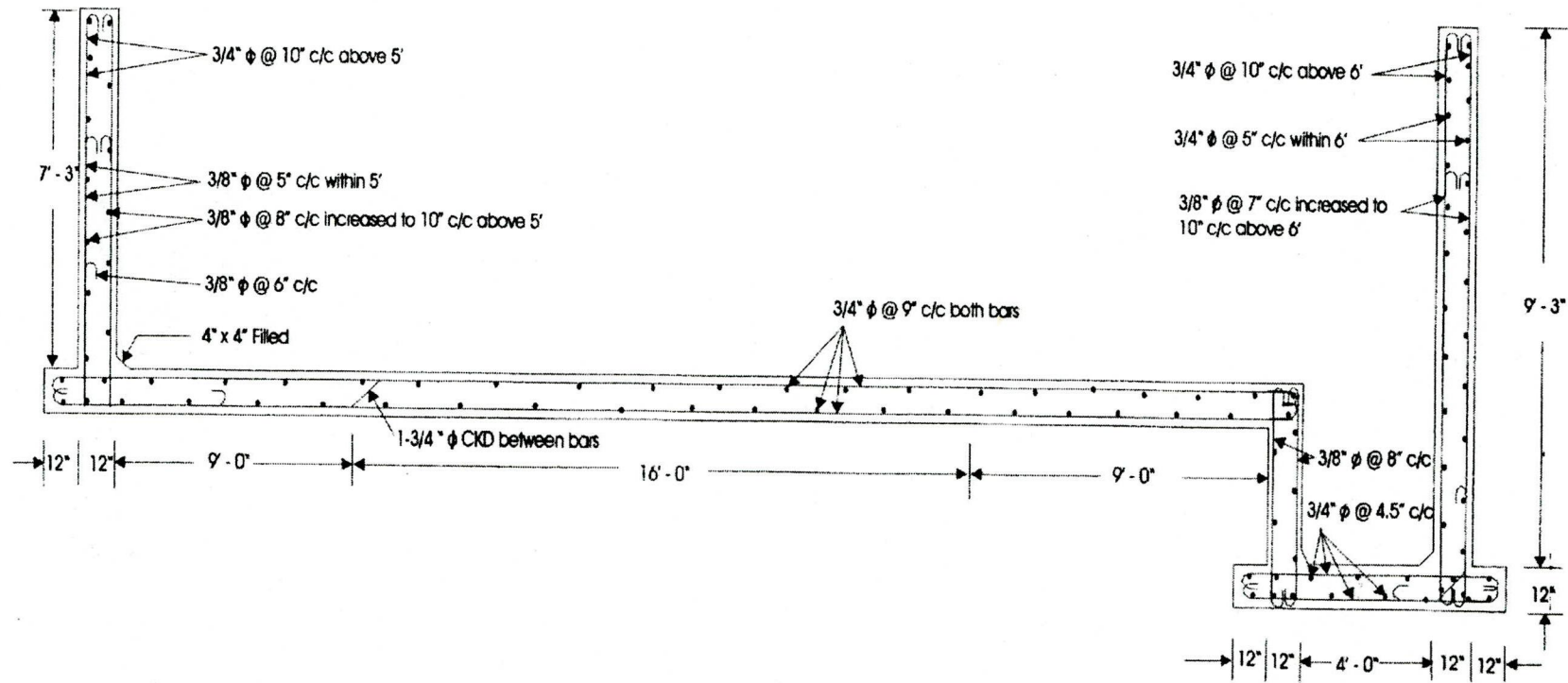


Figure A.6 Section E-E of the final plan of the physical model facility (Source: Sarkar S.M.R, 2001)

APPENDIX B

EQUIVALENT TRAPEZOIDAL CROSS-SECTION

The actual bed levels of the selected reach of the Old-Brahmaputra were undulating in nature. As a result, reproduction of these bed levels for each model run was very hard and time consuming. For this reason, all the cross-sections were changed into equivalent trapezoidal cross-section and then were reproduced in the model for each model run. The plot of actual and equivalent trapezoidal cross-sections for different sections of the study reach is presented below.

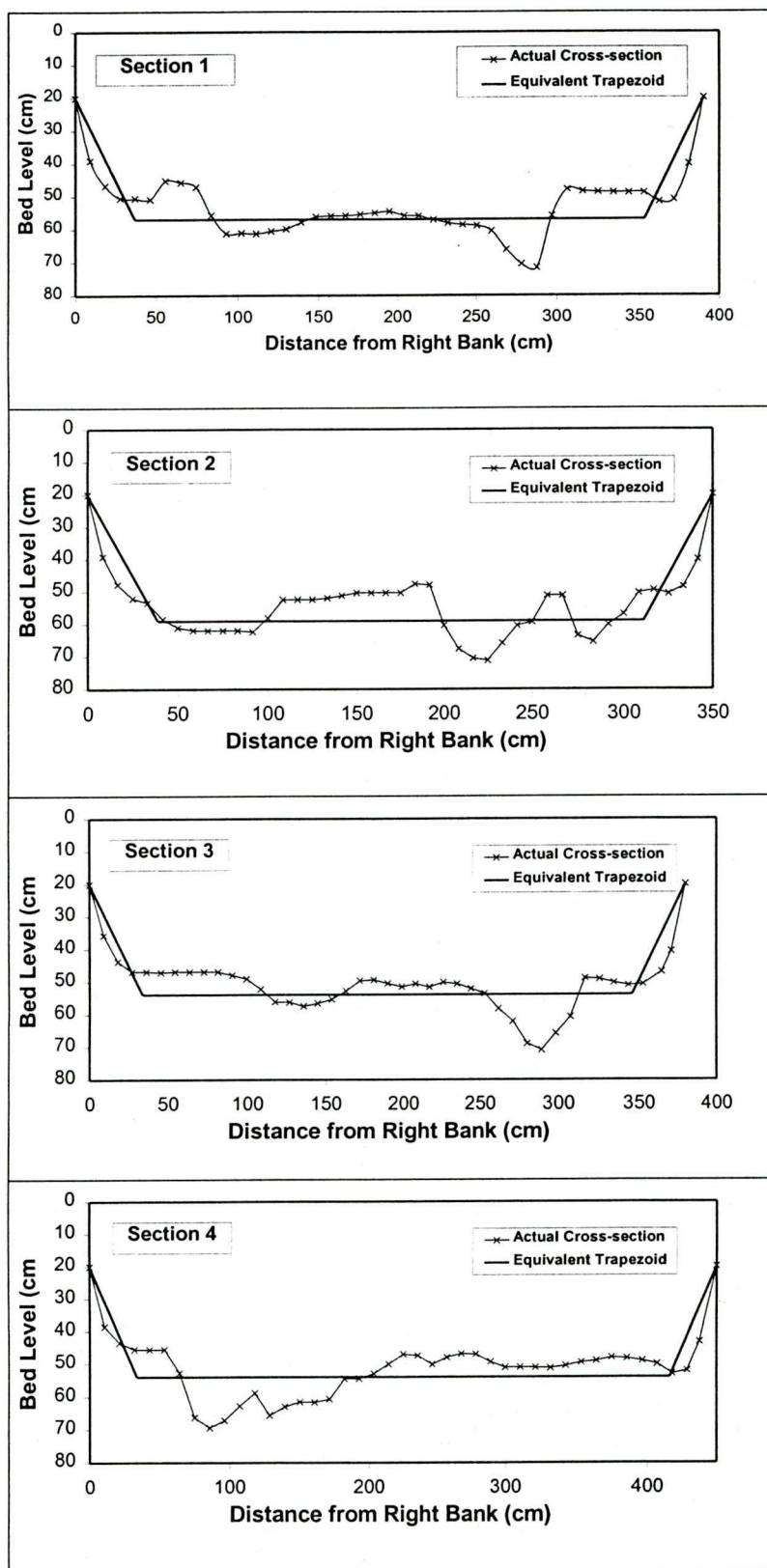


Figure. B.1 Actual cross-section and equivalent trapezoidal cross-section of different sections 1 to 4 of the model reach.

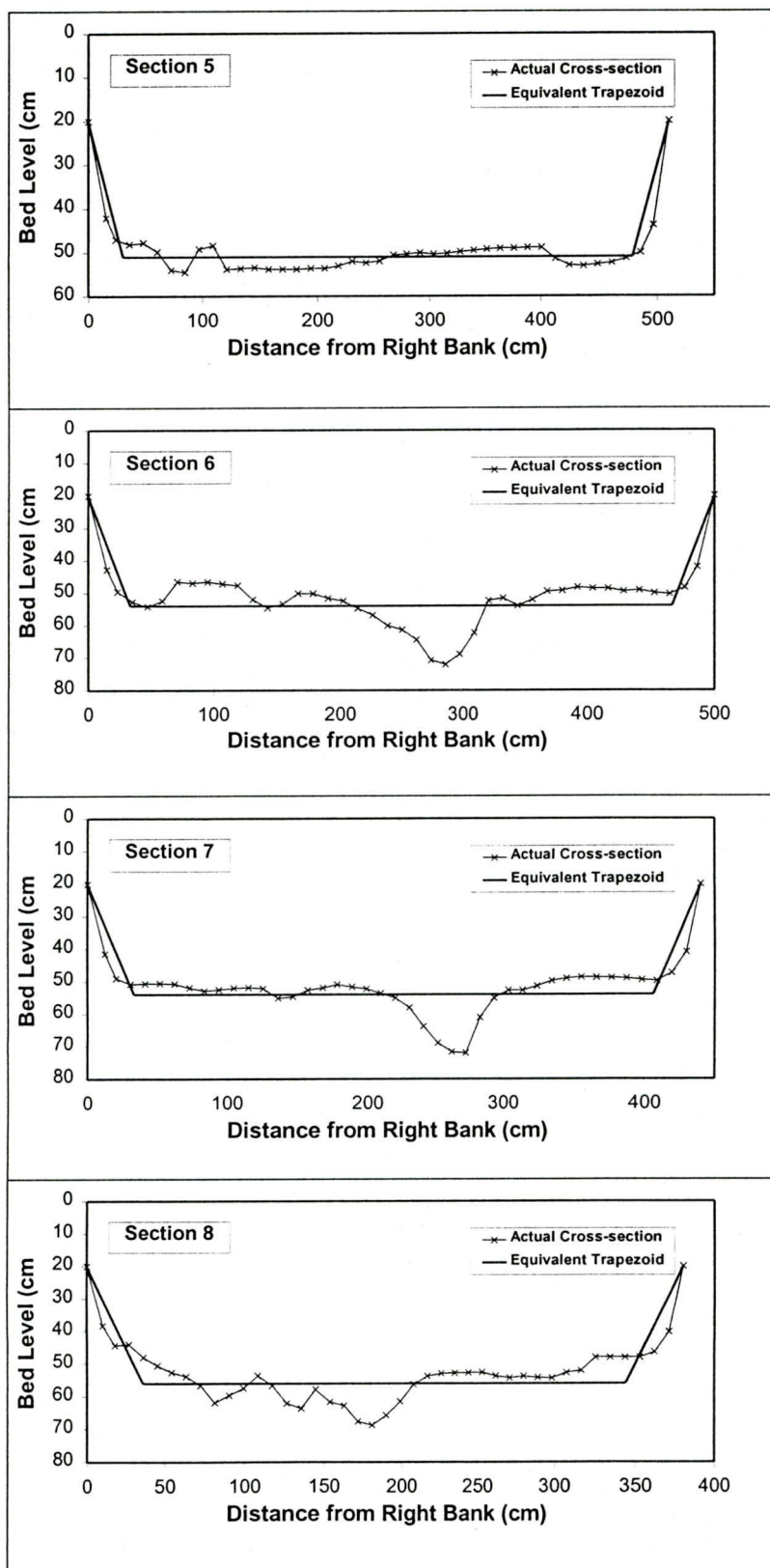


Figure. B.2 Actual cross-section and equivalent trapezoidal cross-section of different sections 5 to 8 of the model reach.

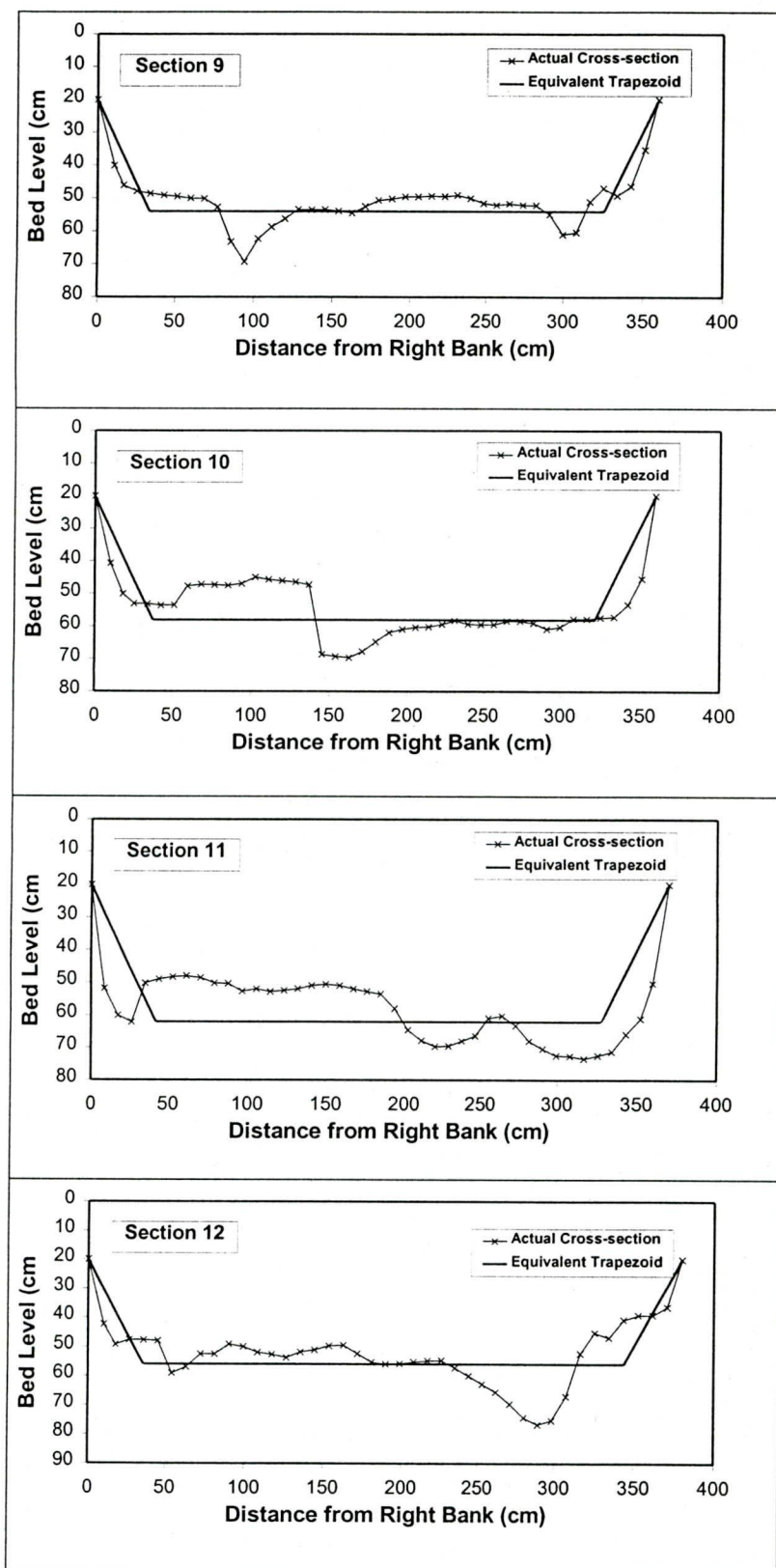


Figure. B.3 Actual cross-section and equivalent trapezoidal cross-section of different sections 9 to 12 of the model reach.

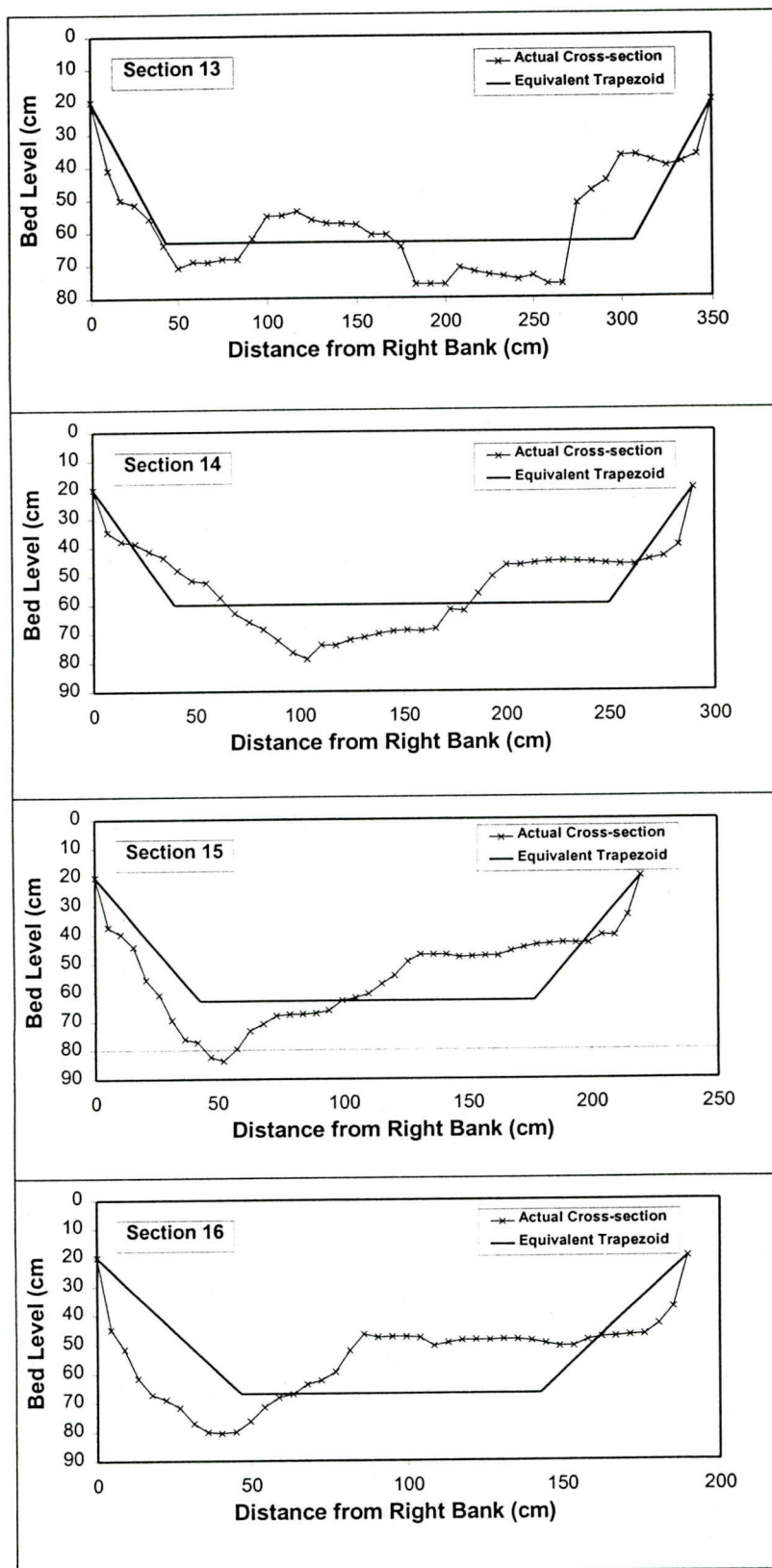


Figure. B.4 Actual cross-section and equivalent trapezoidal cross-section of different sections 13 to 16 of the model reach.

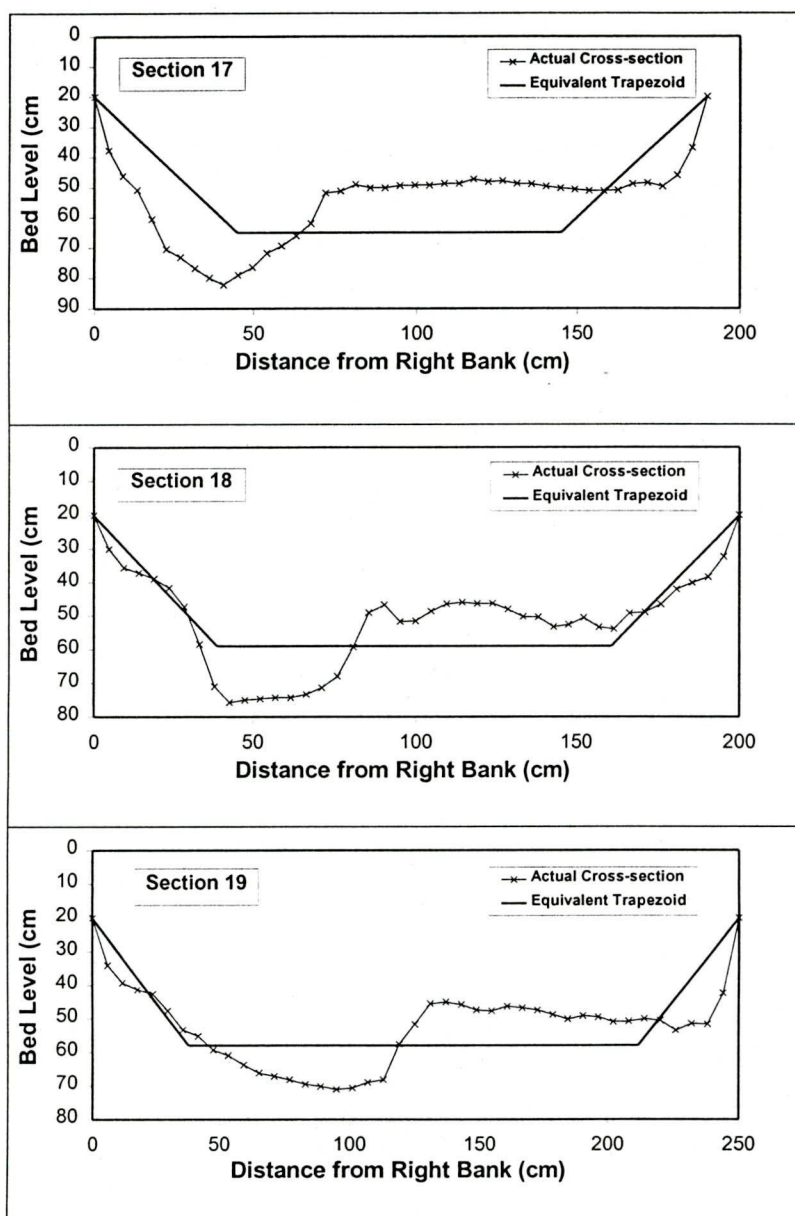
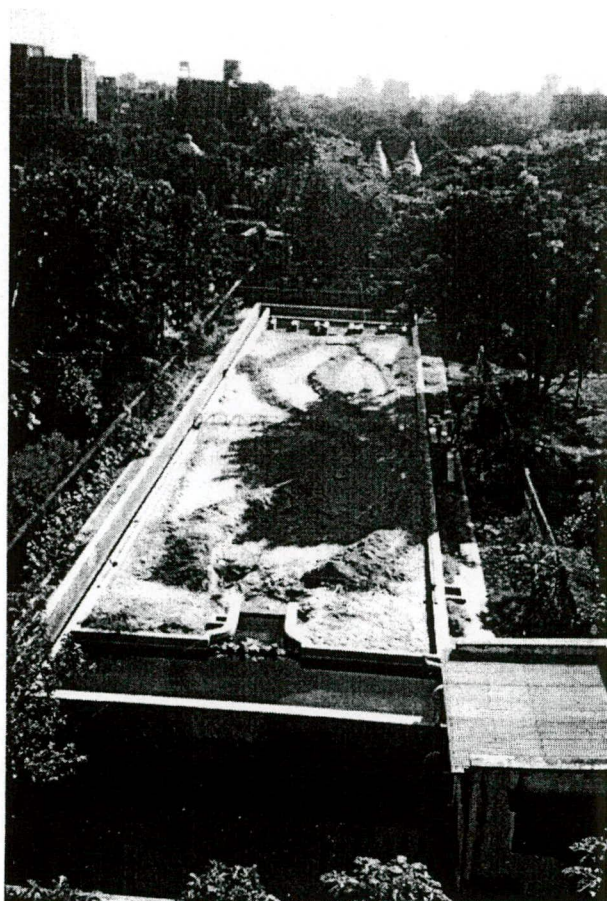


Figure. B.5 Actual cross-section and equivalent trapezoidal cross-section of different sections 17 to 19 of the model reach.

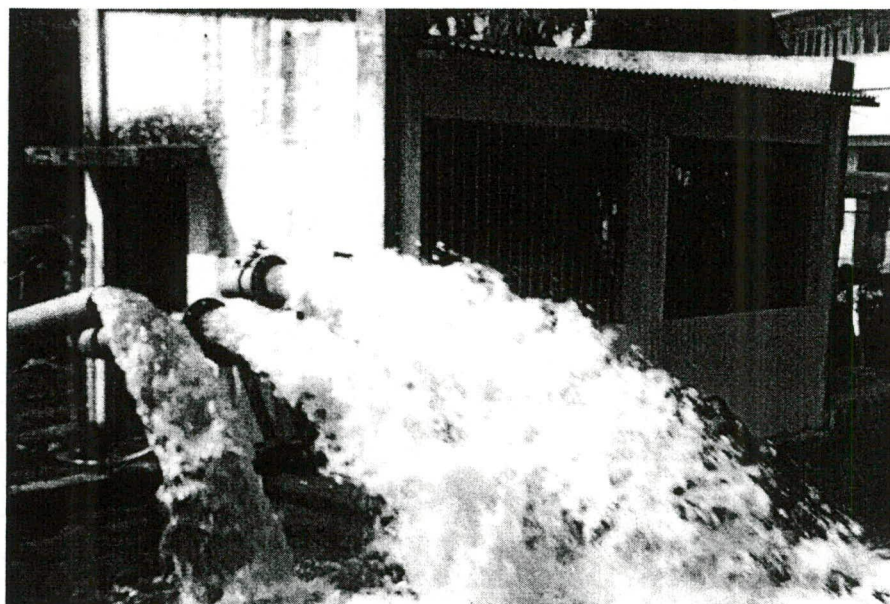
APPENDIX C

PHOTOGRAPHS

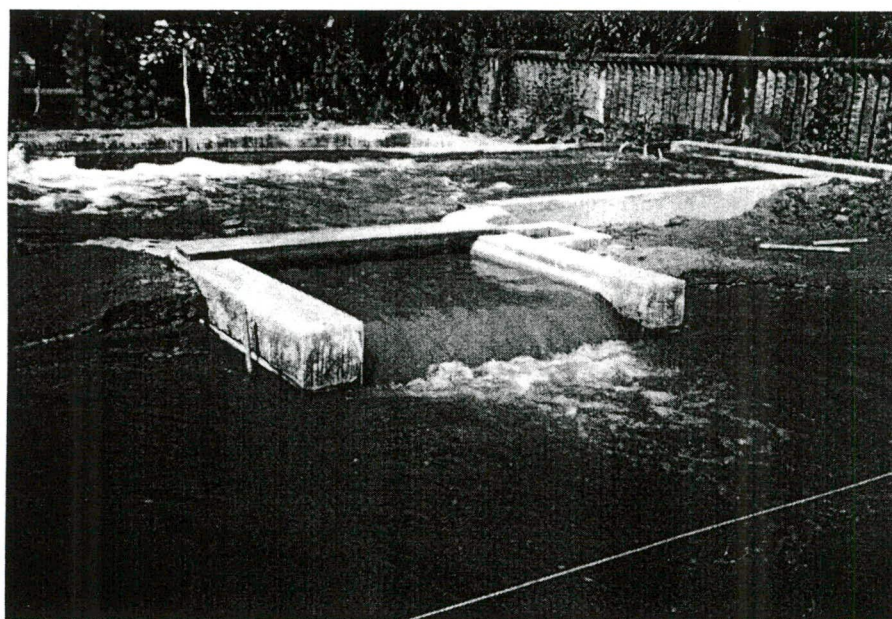
Photographs of the physical model taken during the study are presented below:



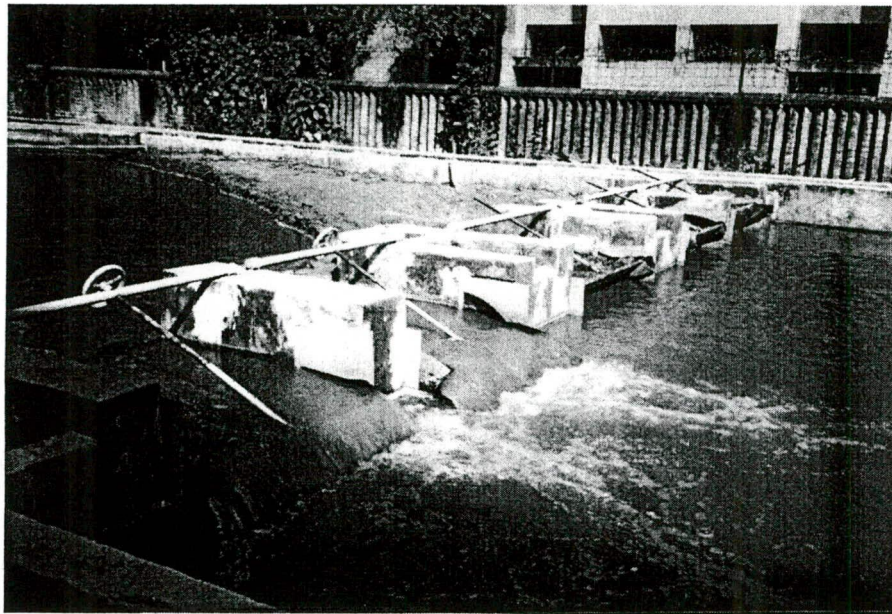
Photograph C.1 Top view of the Physical Model facility



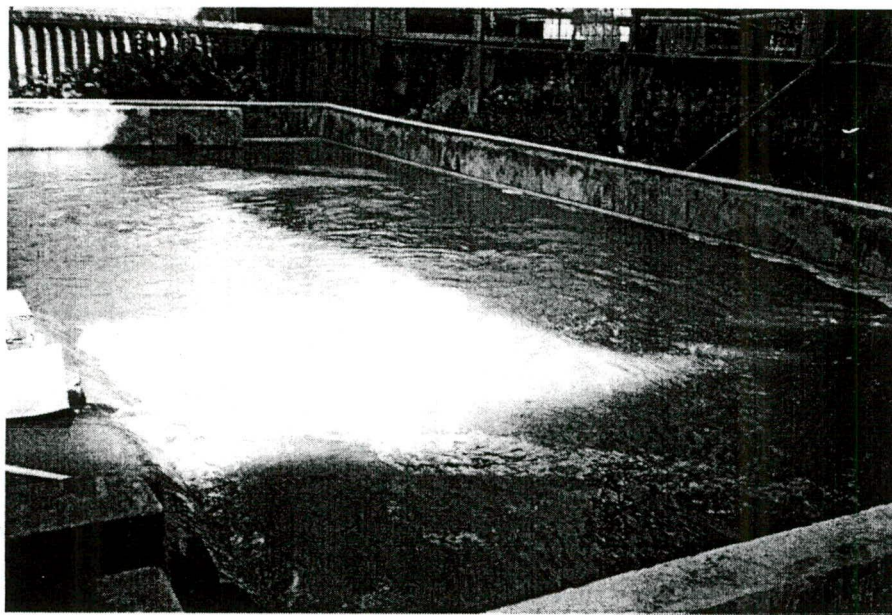
Photograph C.2 Pumps and the Pump House



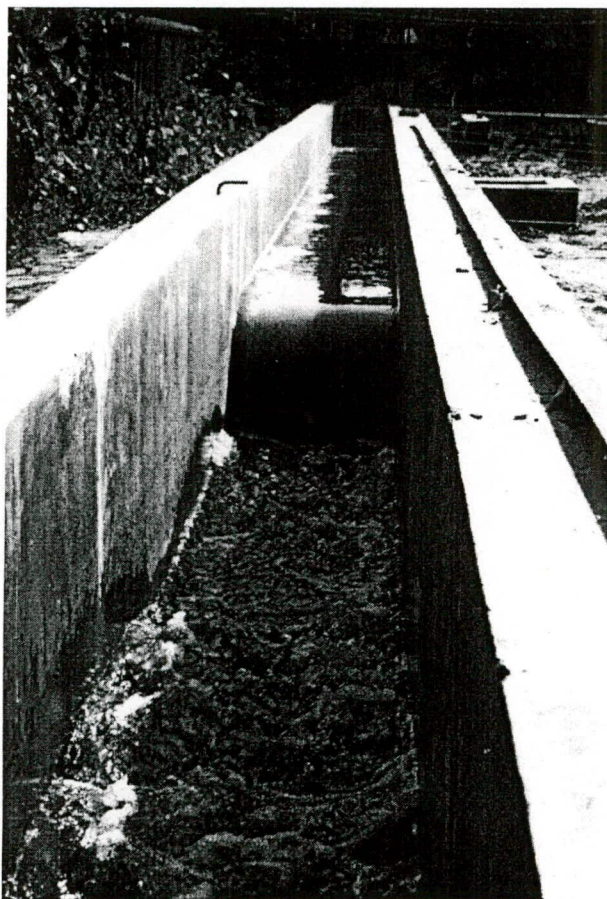
Photograph C.3 Upstream Reservoir and Rehbock Weir-1



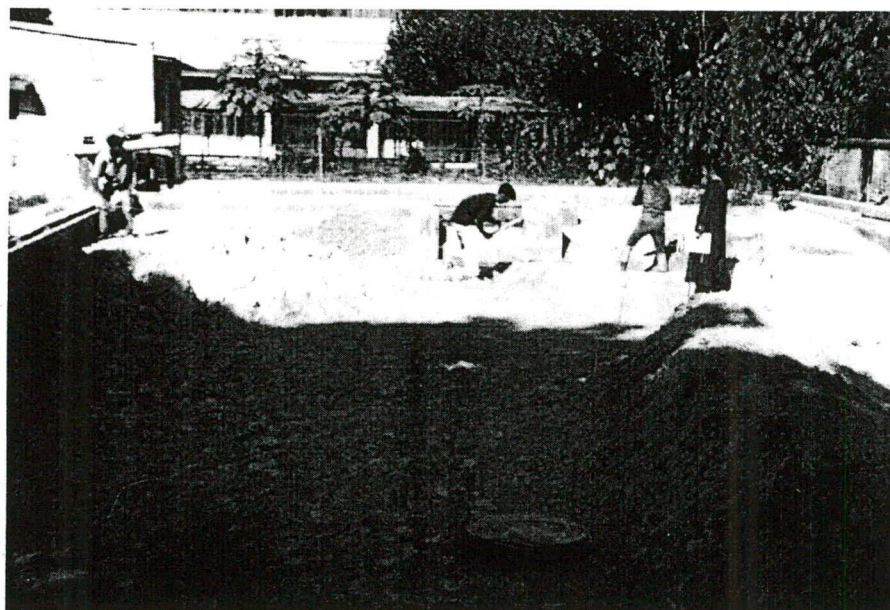
Photograph C.4 Tail Gates and Sediment Trap Tank



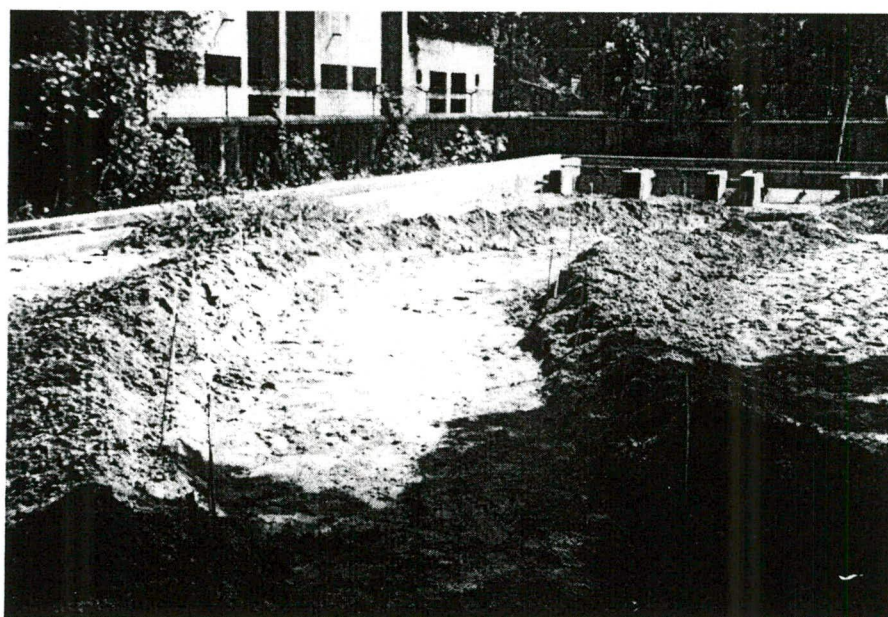
Photograph C.5 Downstream Reservoir



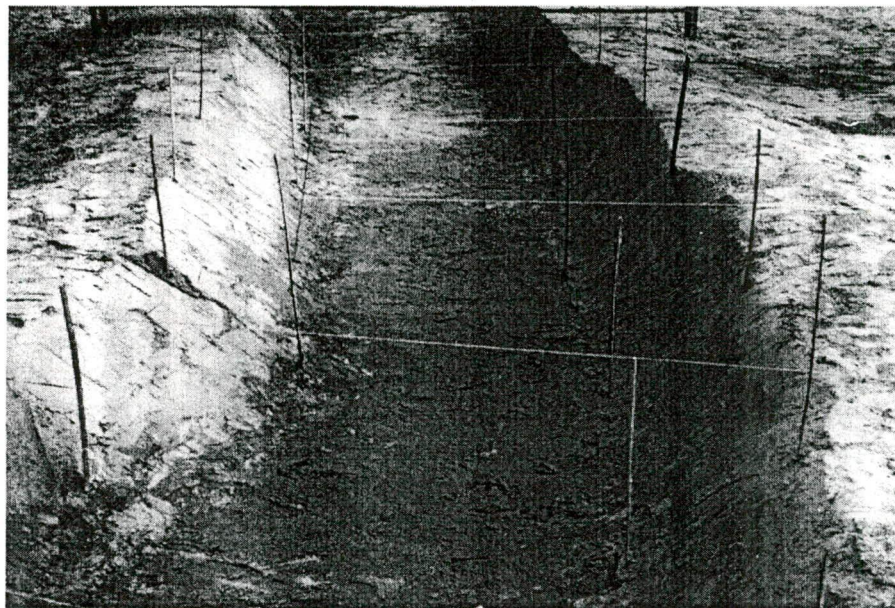
Photograph C.6 Re-circulating Canal and Rehbock Weir-2



Photograph C.7 Earthwork during model setup



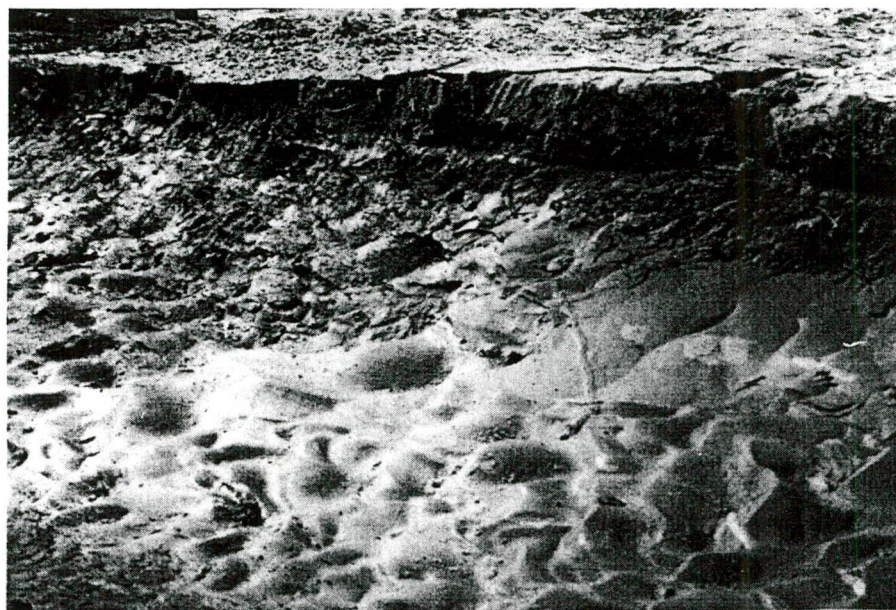
Photograph C.8 Layout setting of model river reach



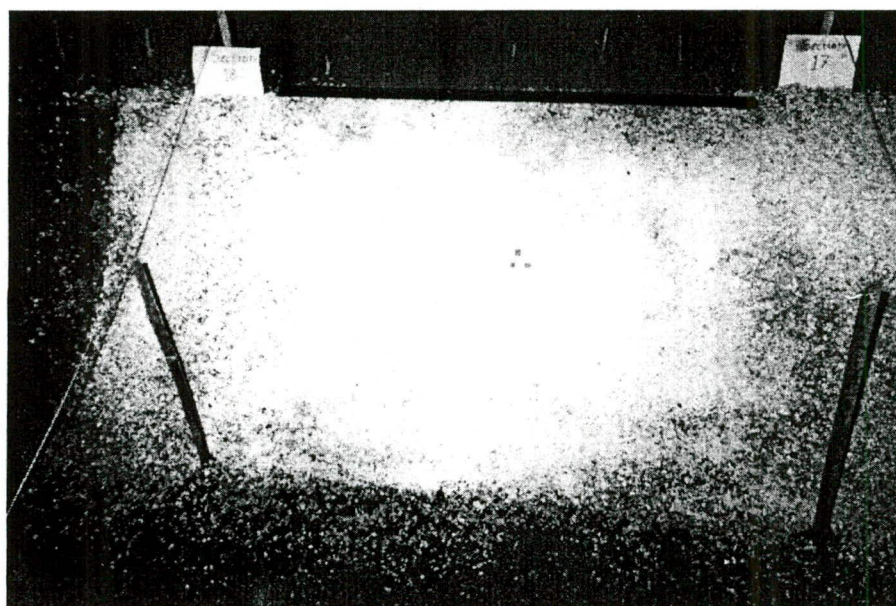
Photograph C.9 Setting out Trapezoidal cross-sections



Photograph C.10 Model run for calibration



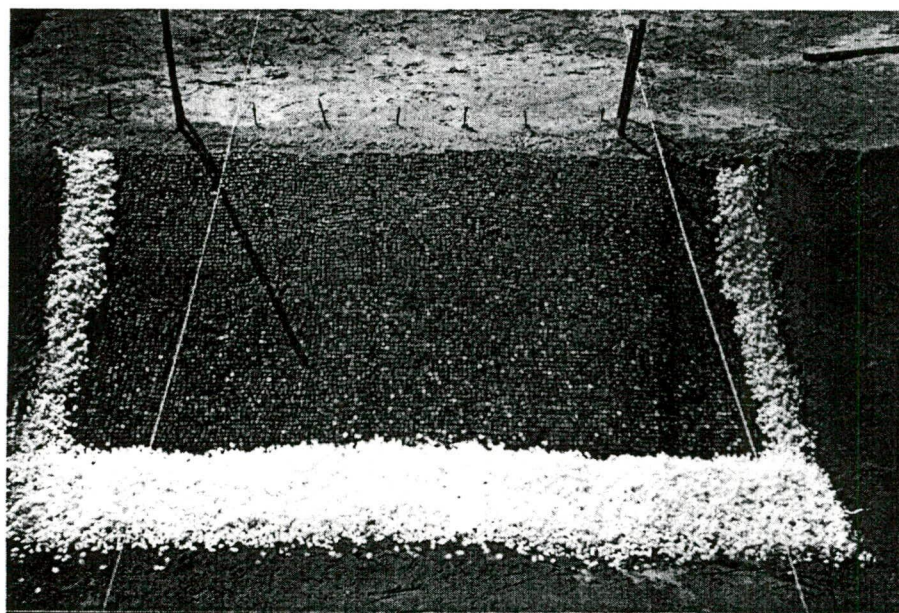
Photograph C.11 Bank erosion at the study reach



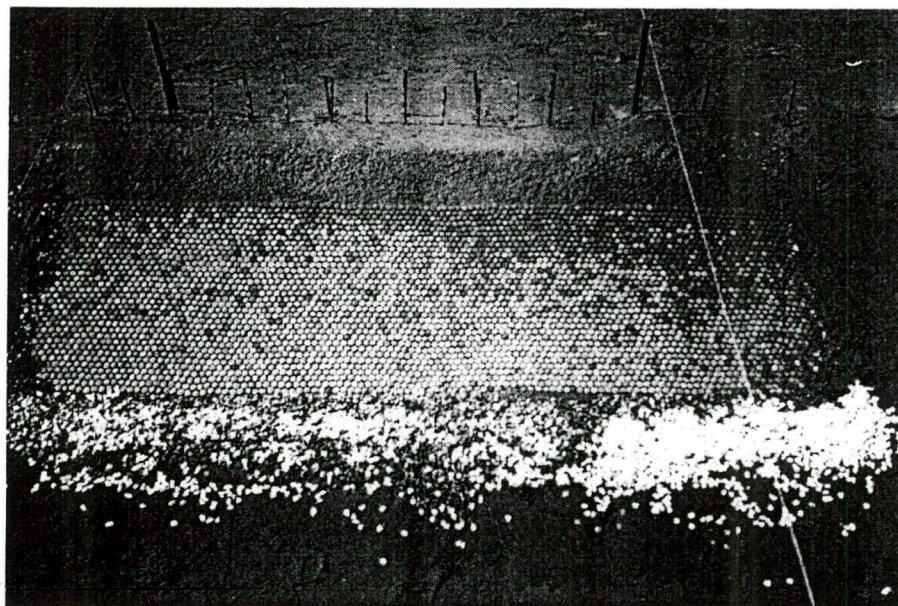
Photograph C.12 Filter layer of the bank protection work



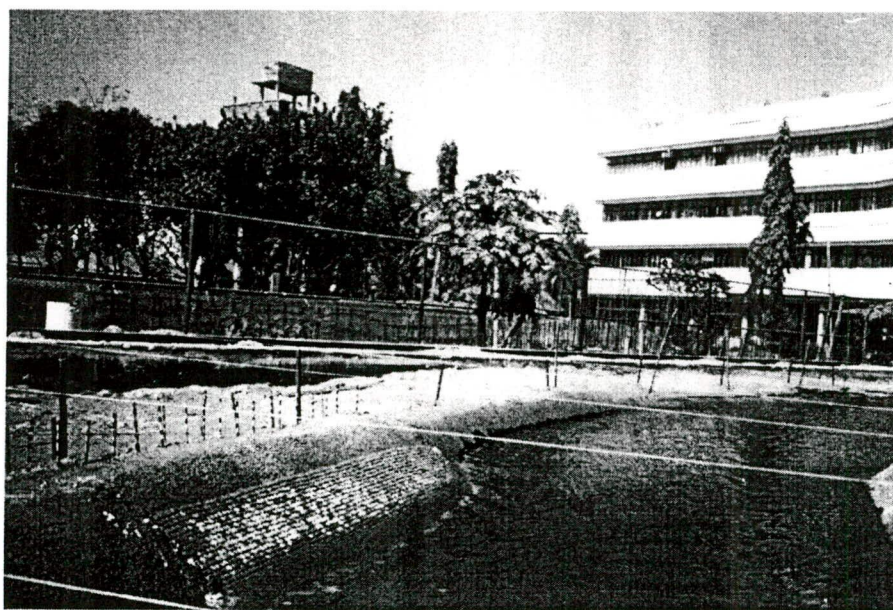
Photograph C.13 Placing blocks on the slope of bank protection



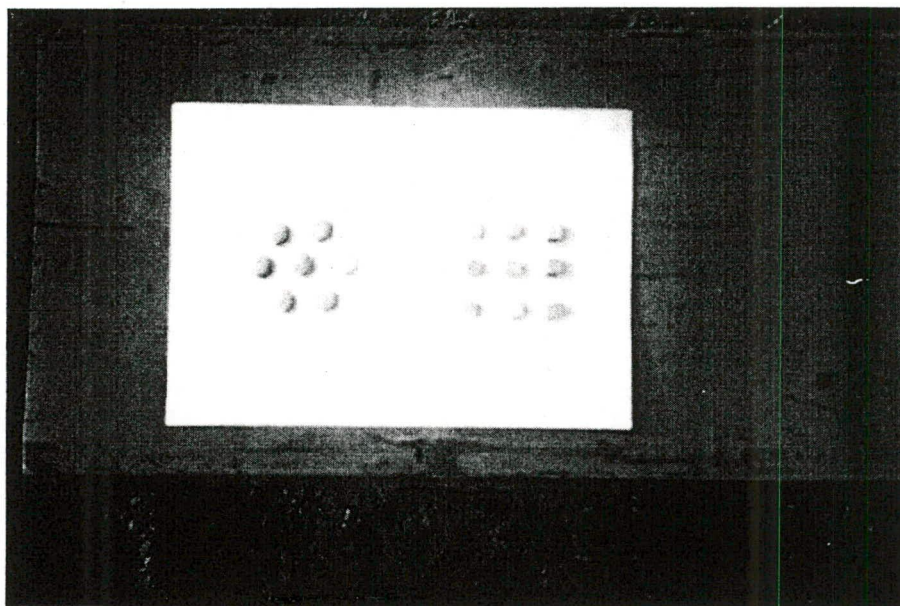
Photograph C.14 Bank protection work with cubical blocks



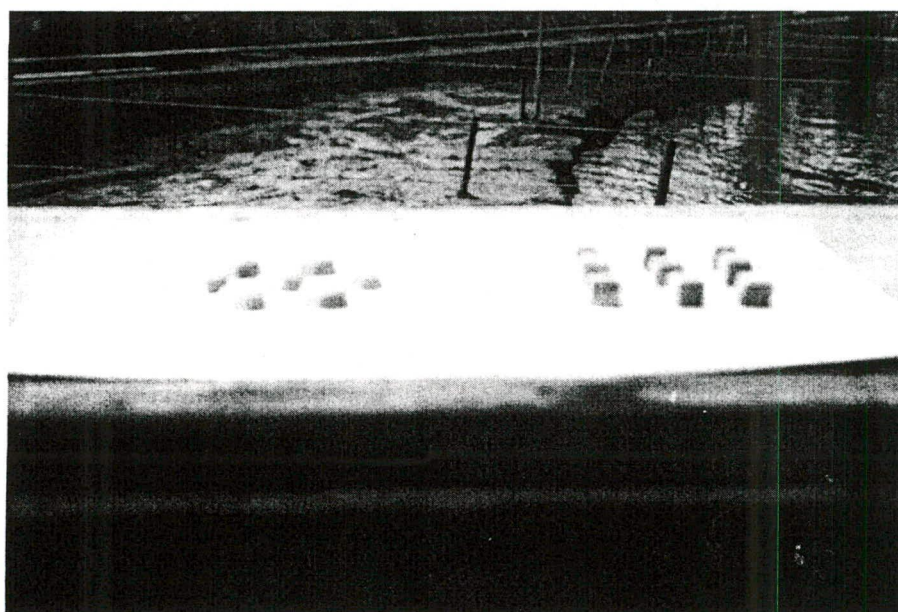
Photograph C.15 Bank protection work with hexagonal blocks



Photograph C.16 During model run



Photograph C.17 Top view of Cubical and Hexagonal revetment blocks



Photograph C.18 Side view of Cubical and Hexagonal revetment blocks

APPENDIX D

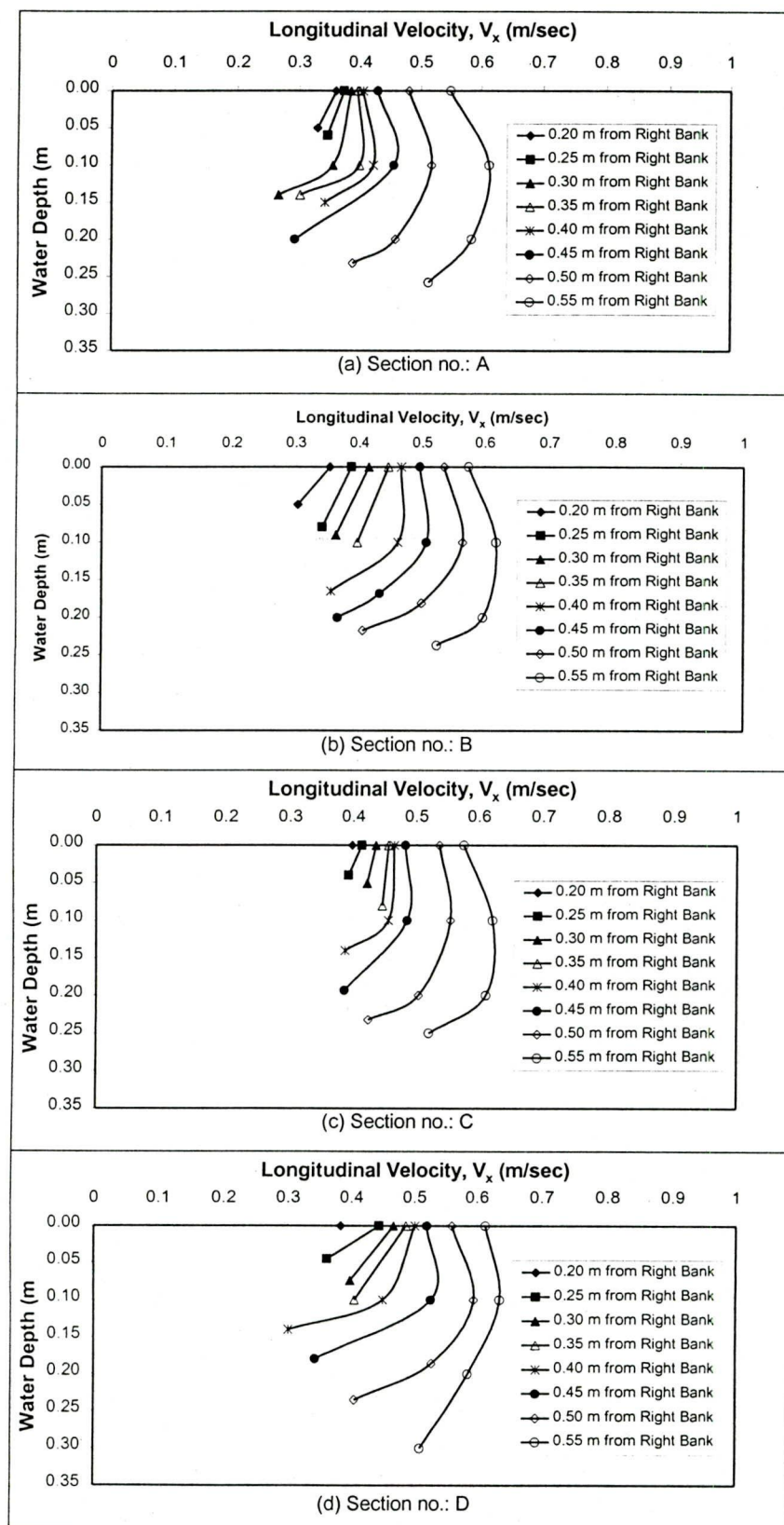


Fig. D.1 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections A to D for cubical type revetments with 2:1 bank slope

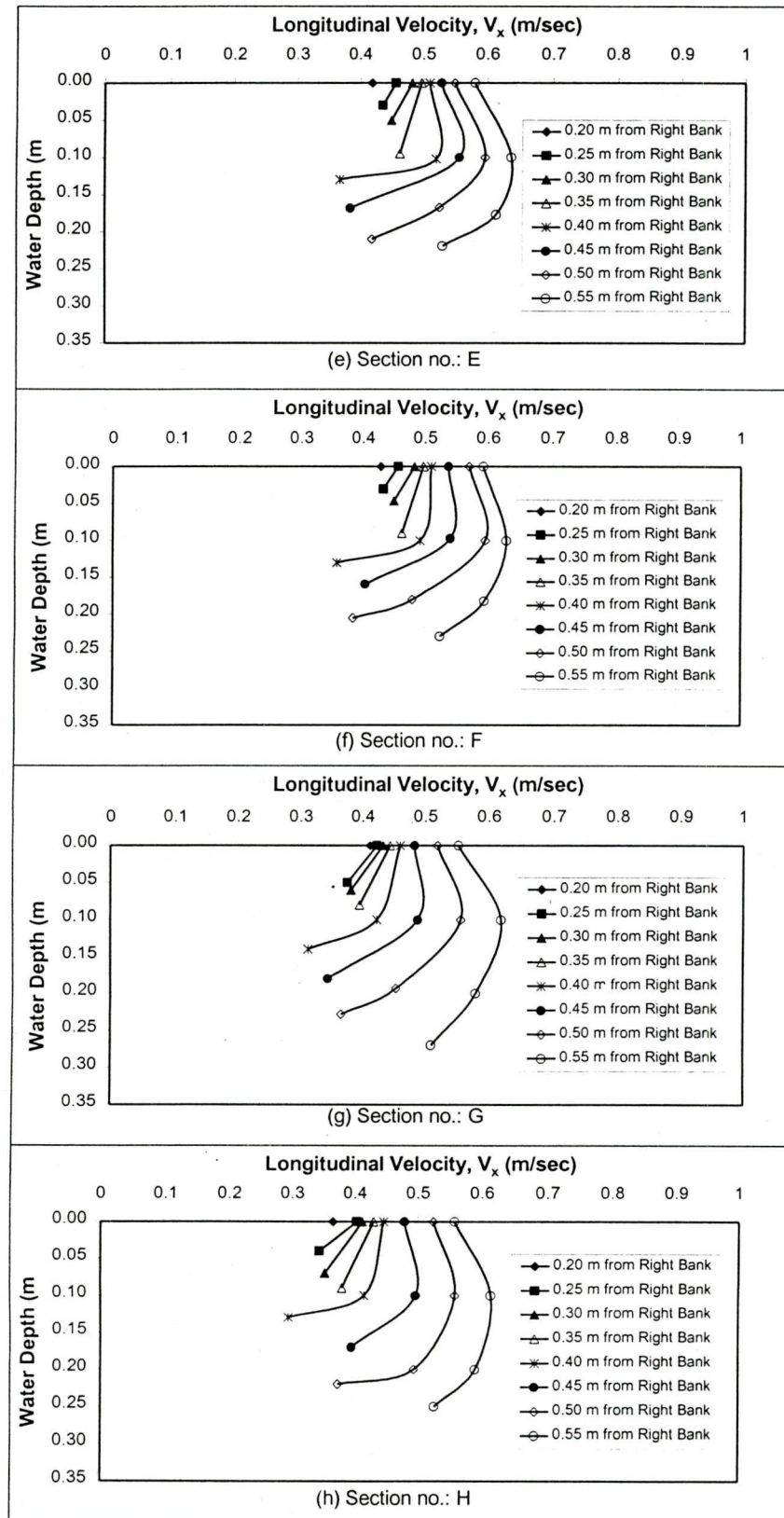


Fig. D.2 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections E to H for cubical type revetments with 2:1 bank slope

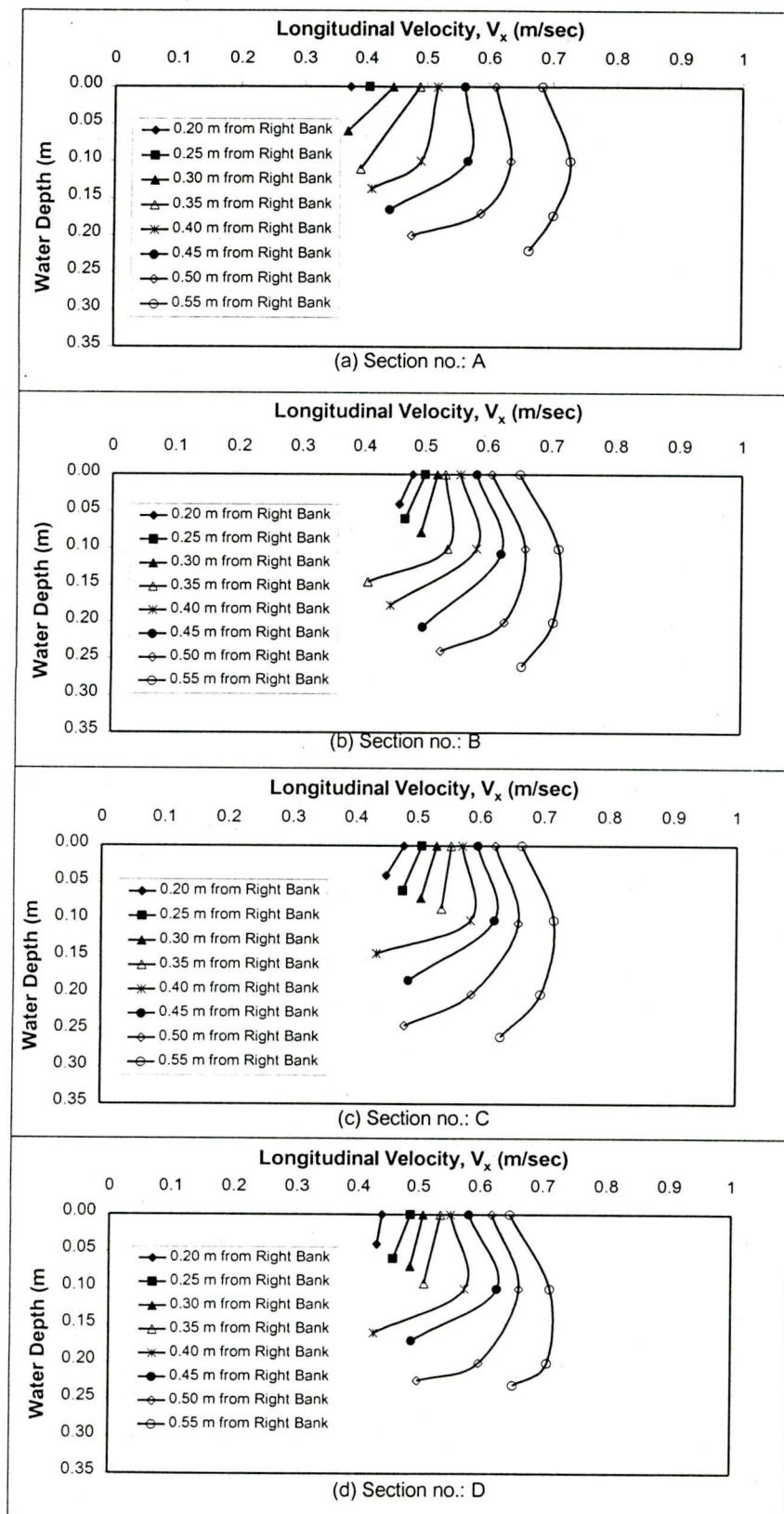


Fig. D.3 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections A to D for cubical type revetments with 2:1 bank slope

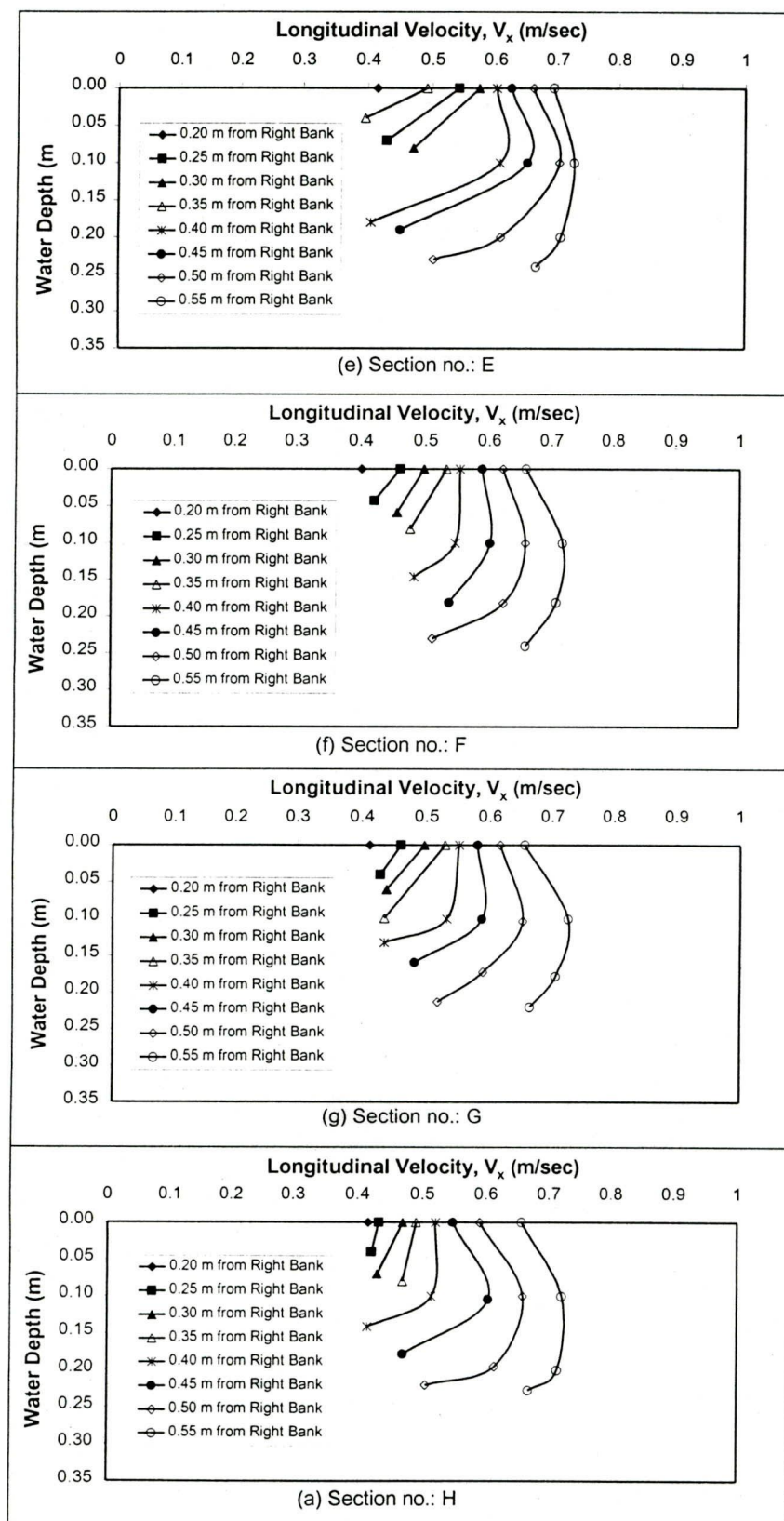


Fig. D.4 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections E to H for cubical type revetments with 2:1 bank slope

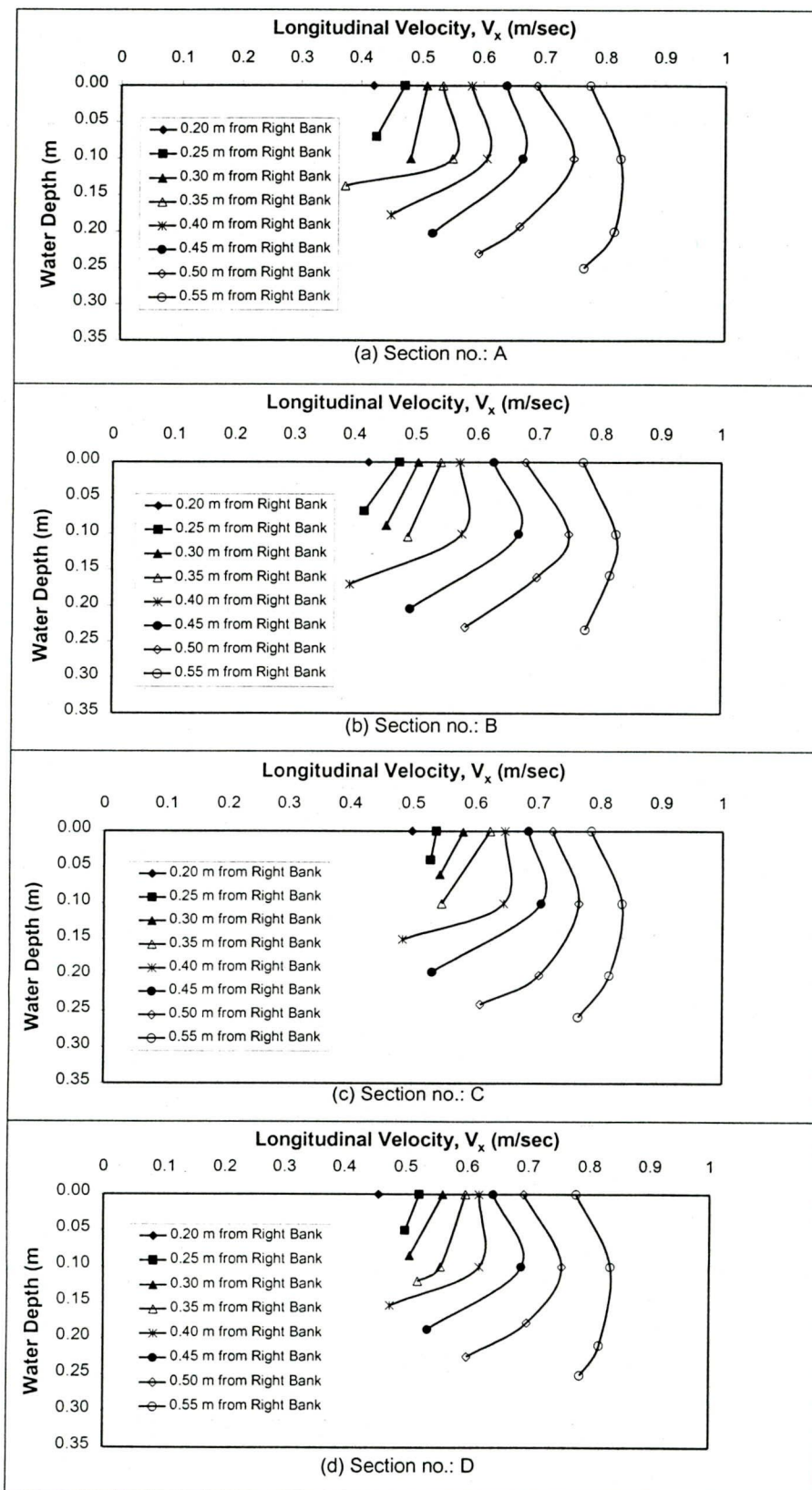


Fig. D.5 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections A to D for cubical type revetments with 2:1 bank slope

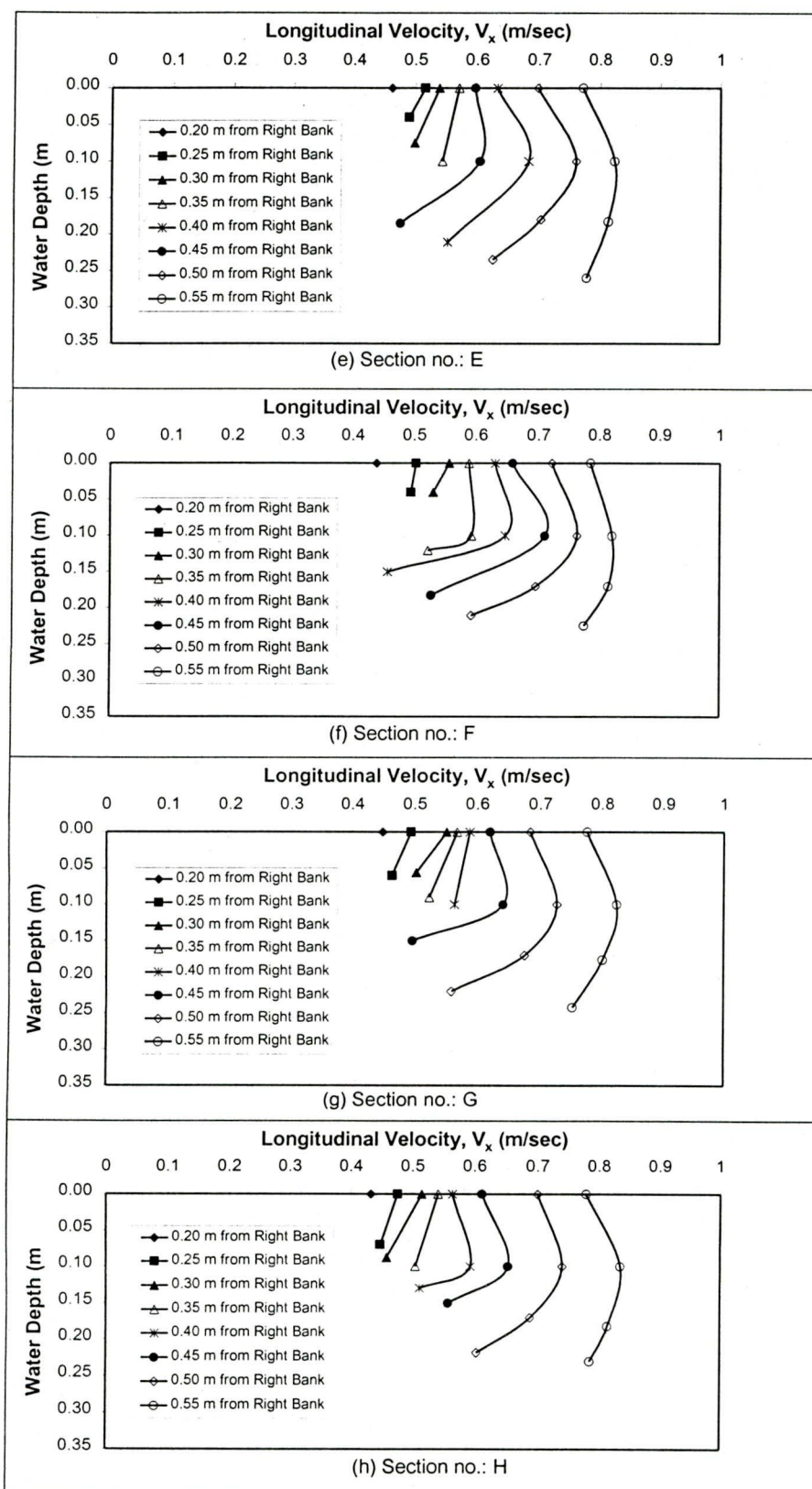


Fig. D.6 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections E to H for cubical type revetments with 2:1 bank slope

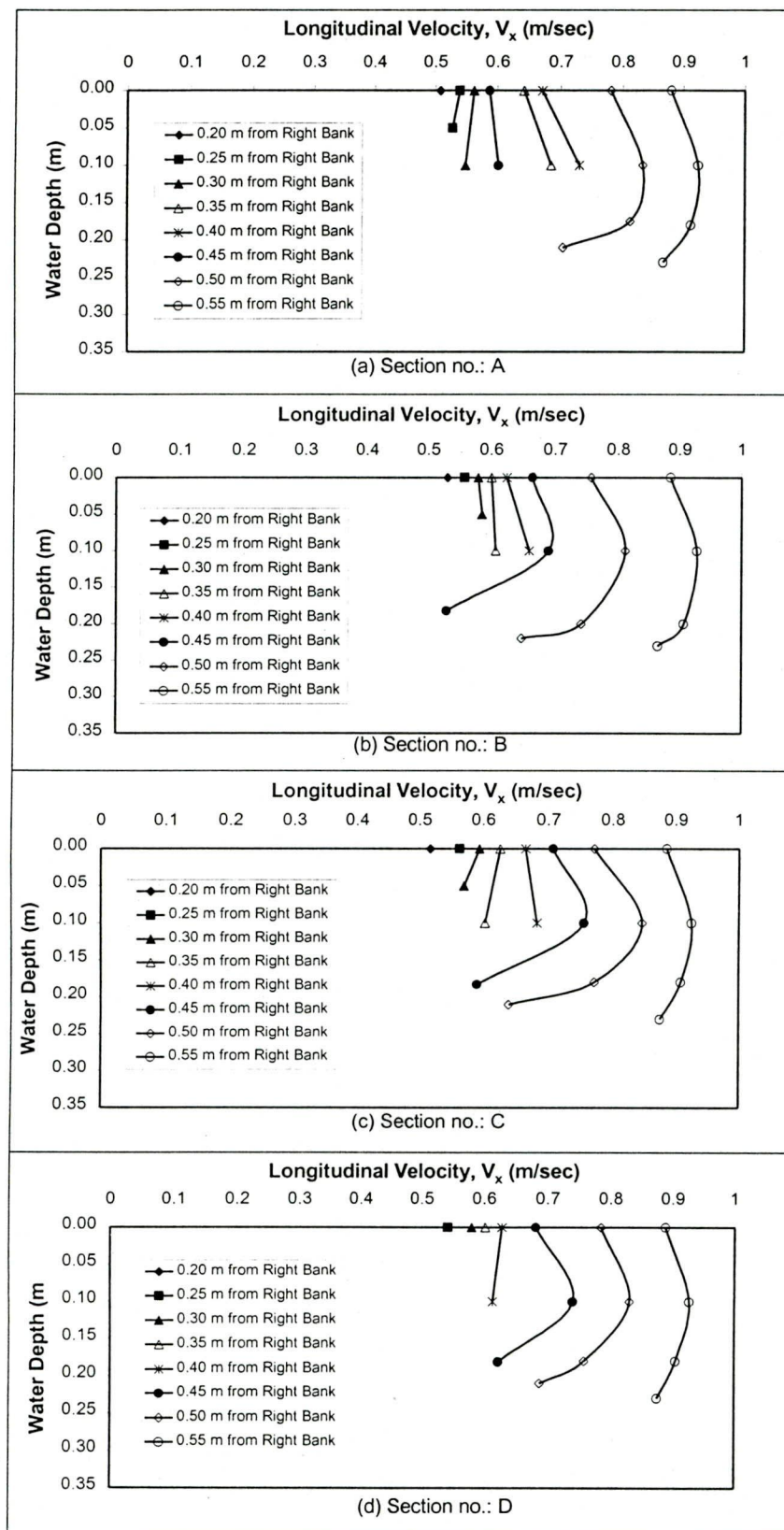


Fig. D.7 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections A to D for cubical type revetments with 2:1 bank slope

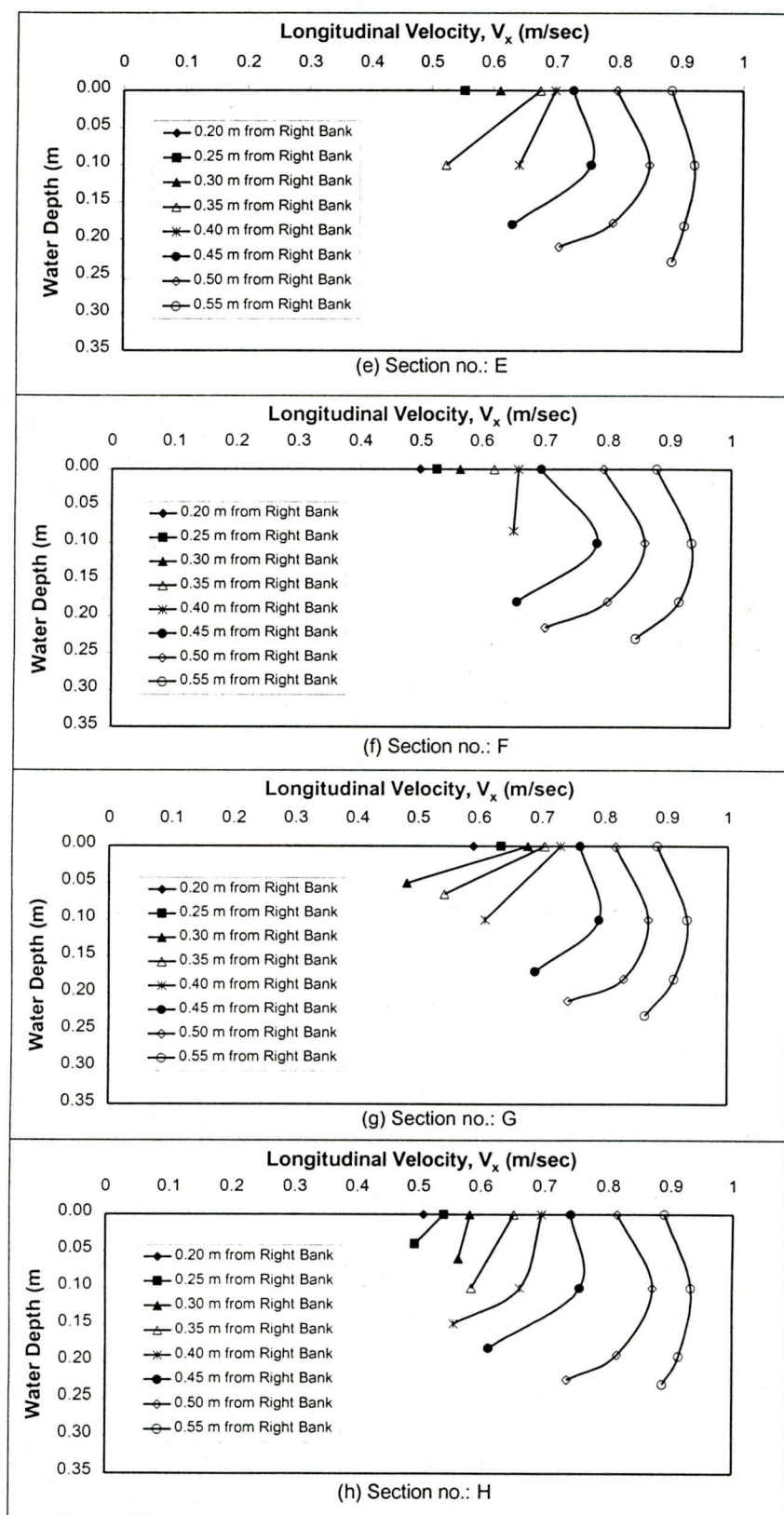


Fig. D.8 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections E to H for cubical type revetments with 2:1 bank slope

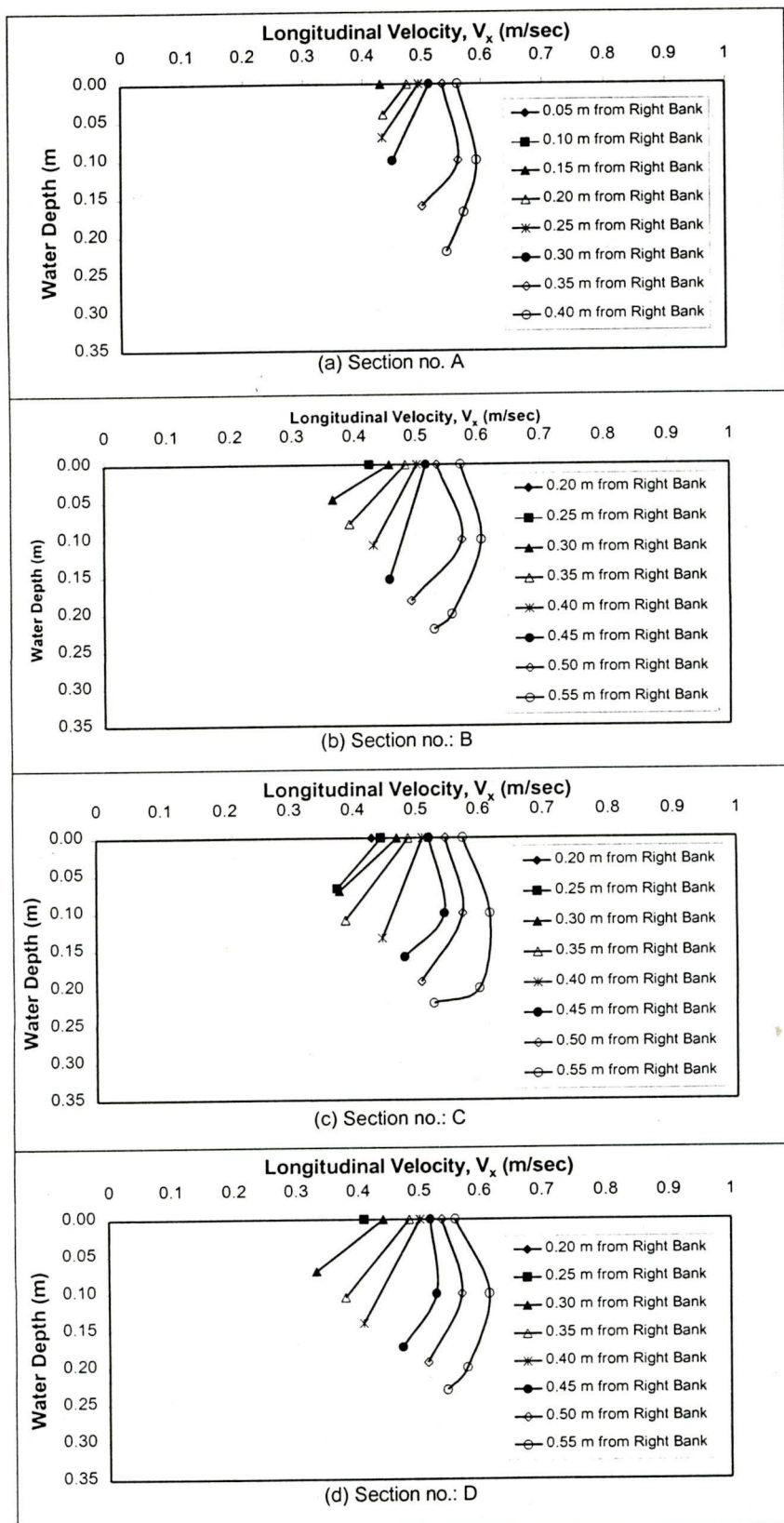


Fig. D.9 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections A to D for cubical type revetments with 3:1 bank slope

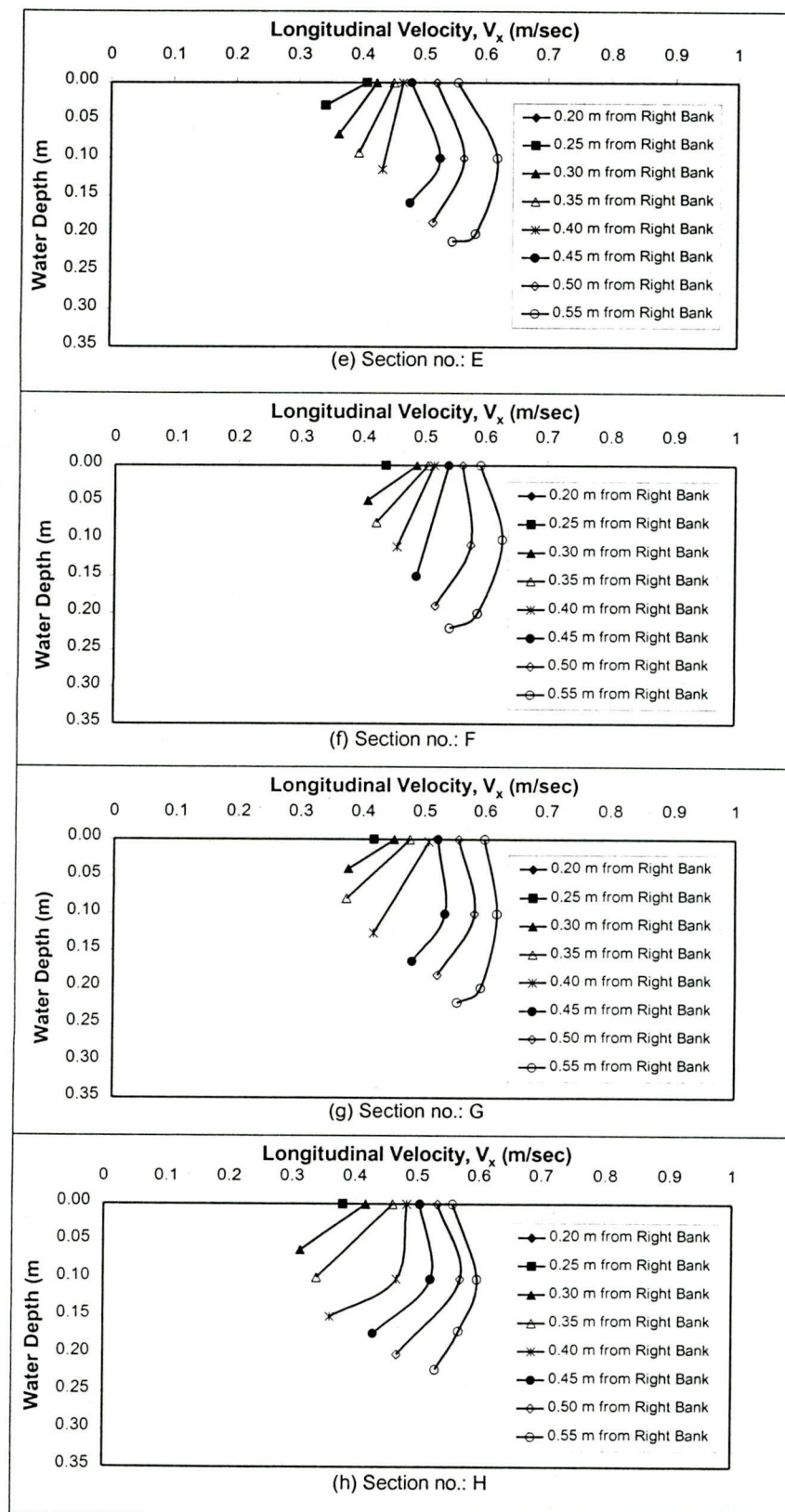


Fig. D.10 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections E to H for cubical type revetments with 3:1 bank slope

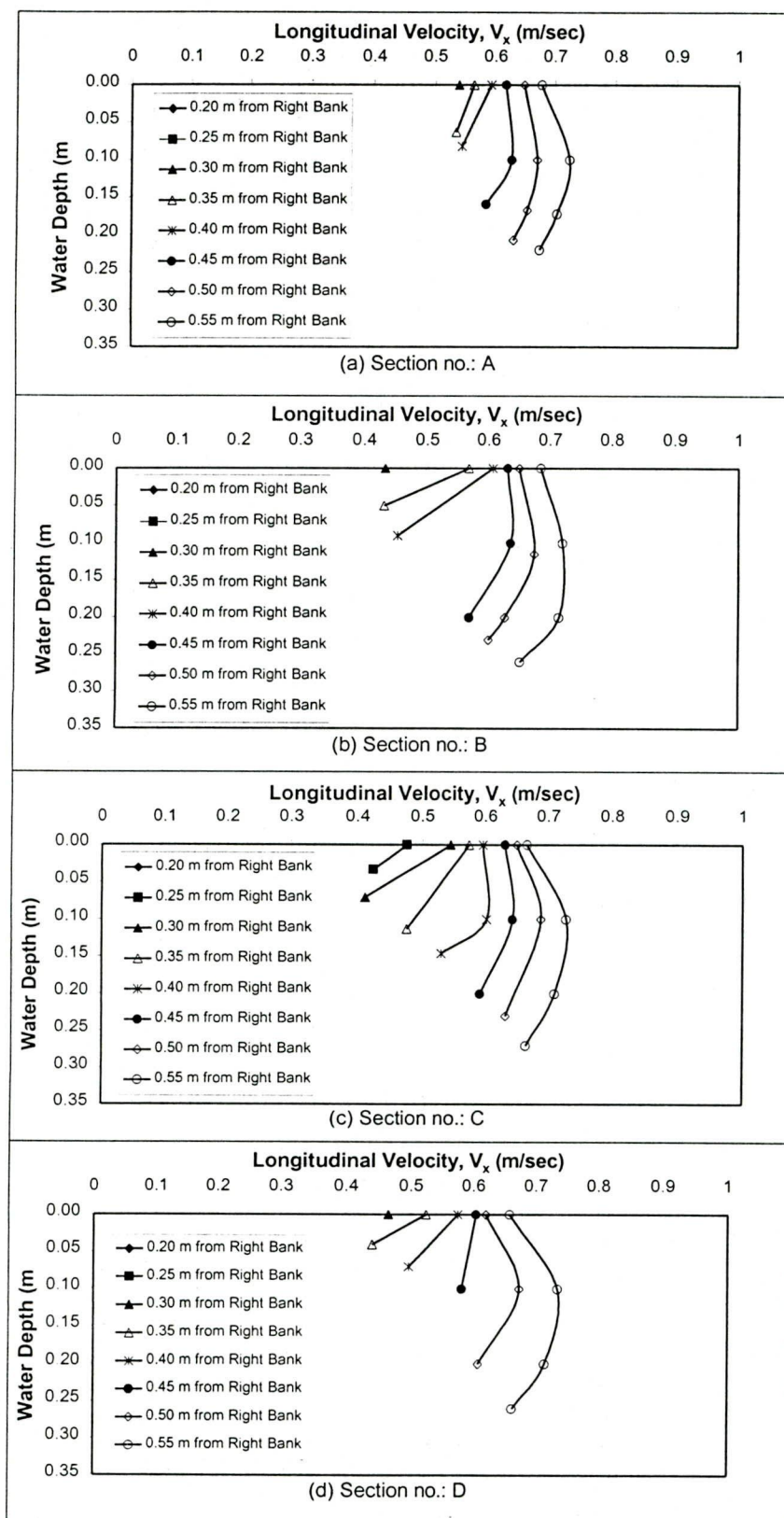


Fig. D.11 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections A to D for cubical type revetments with 3:1 bank slope

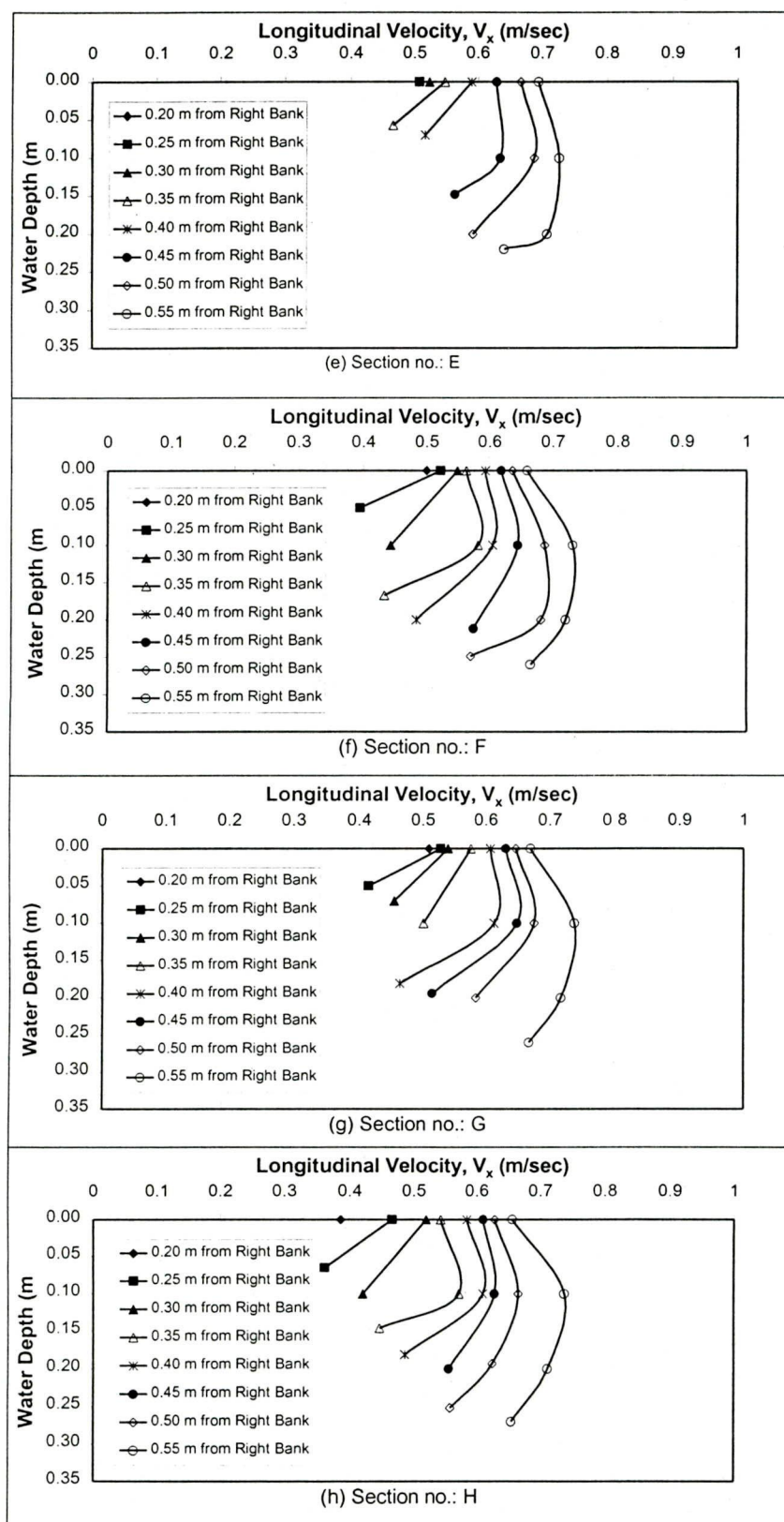


Fig. D.12 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections E to H for cubical type revetments with 3:1 bank slope

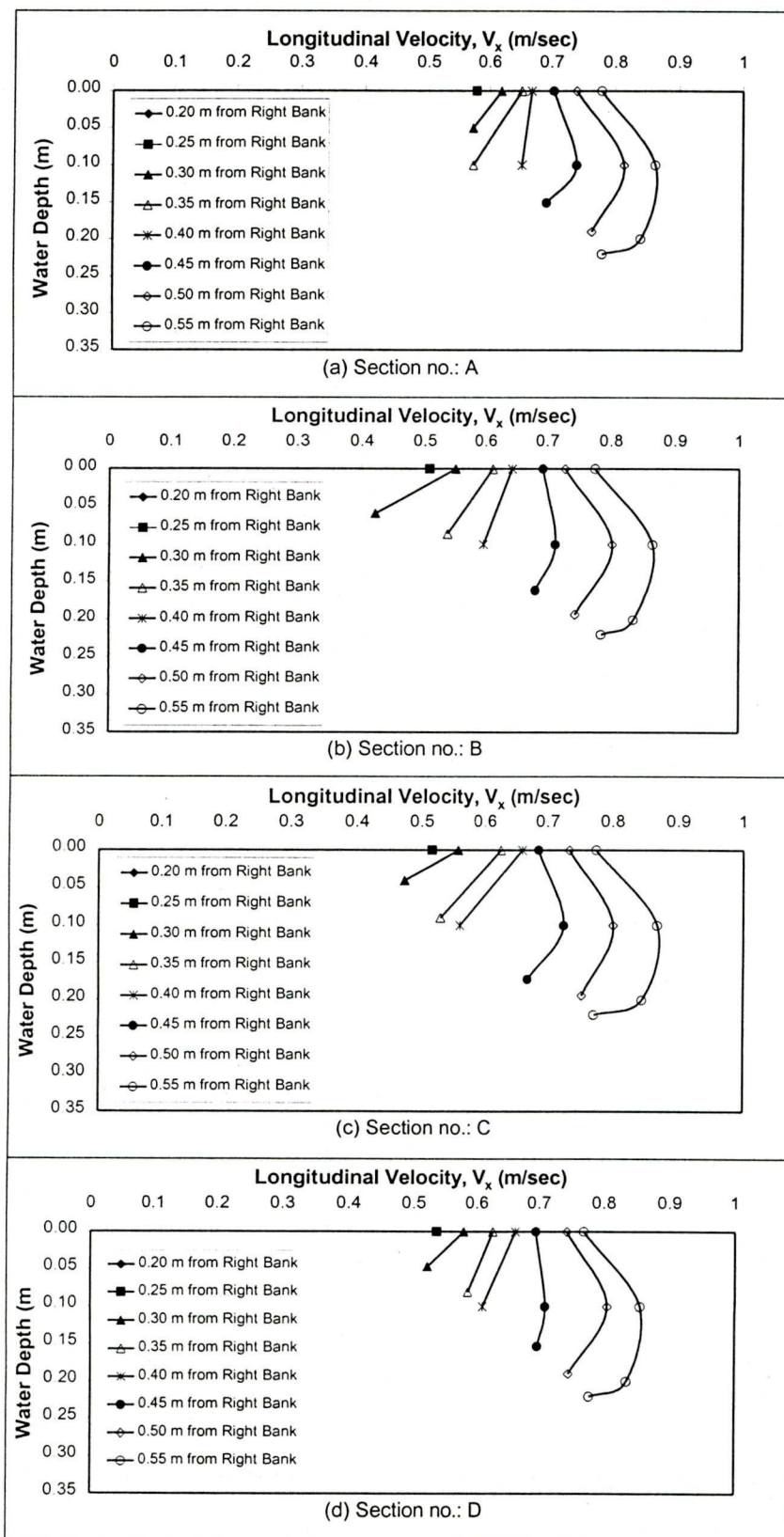


Fig. D.13 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections A to D for cubical type revetments with 3:1 bank slope

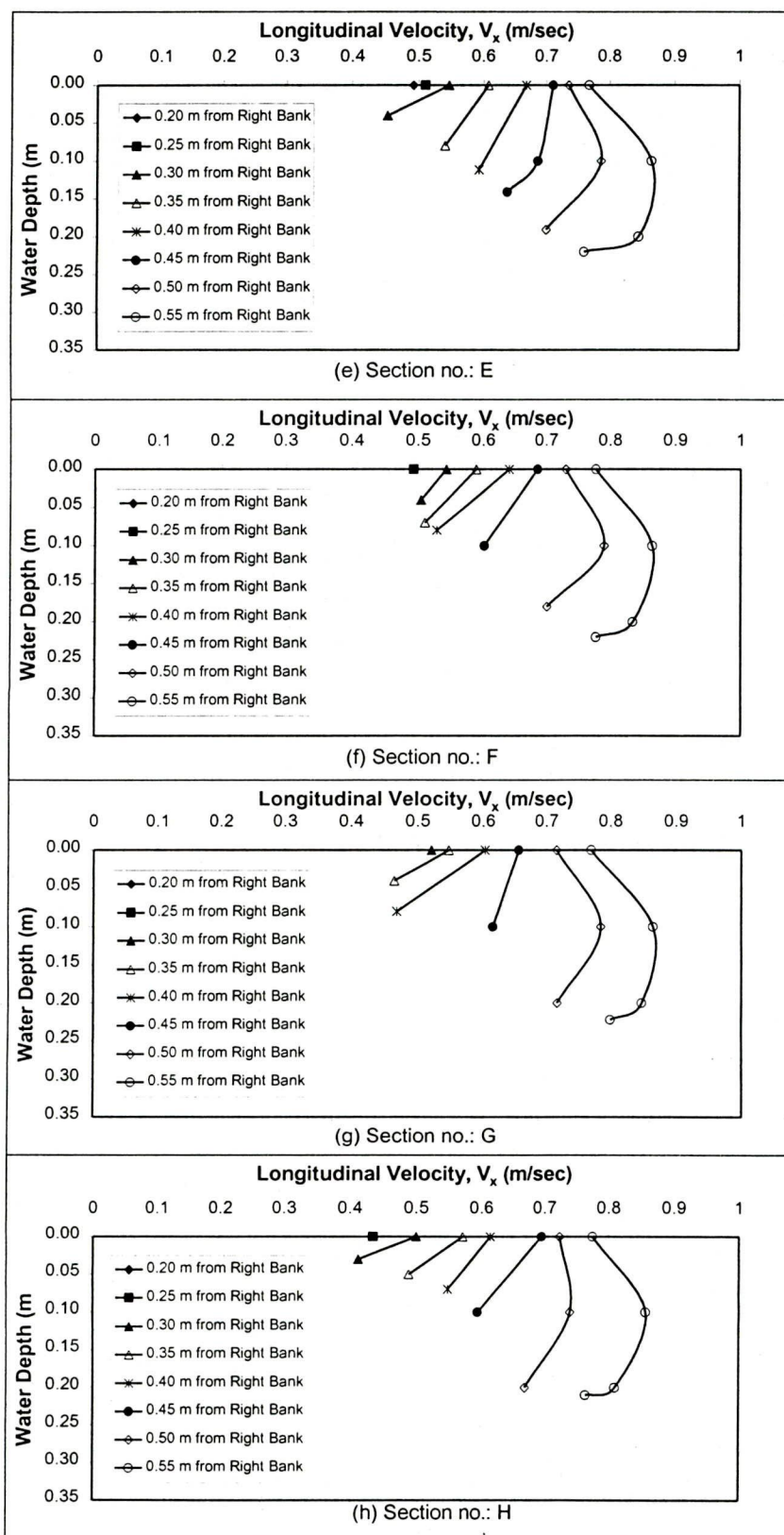


Fig. D.14 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections E to H for cubical type revetments with 3:1 bank slope

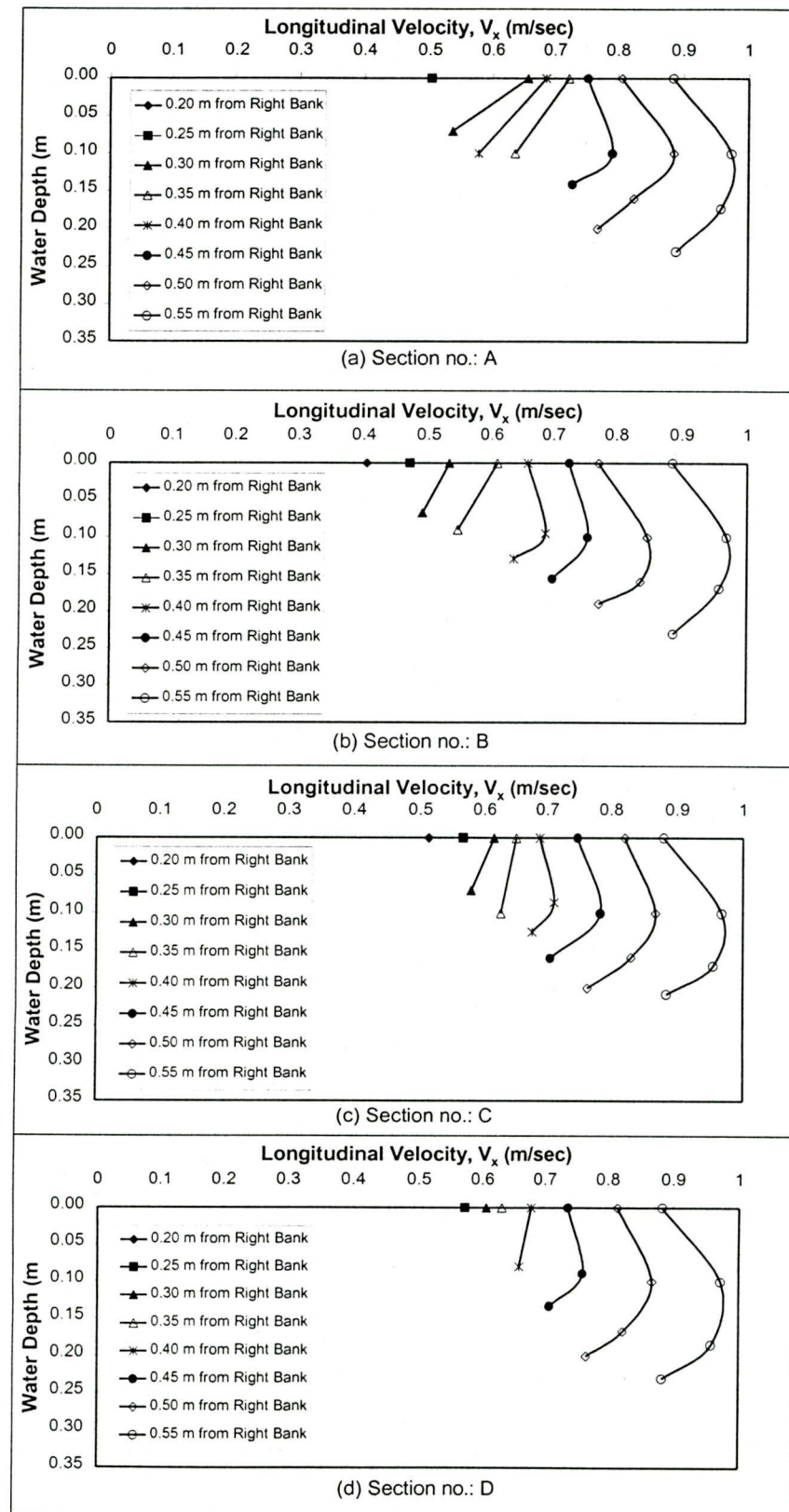


Fig. D.15 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections A to D for cubical type revetments with 3:1 bank slope

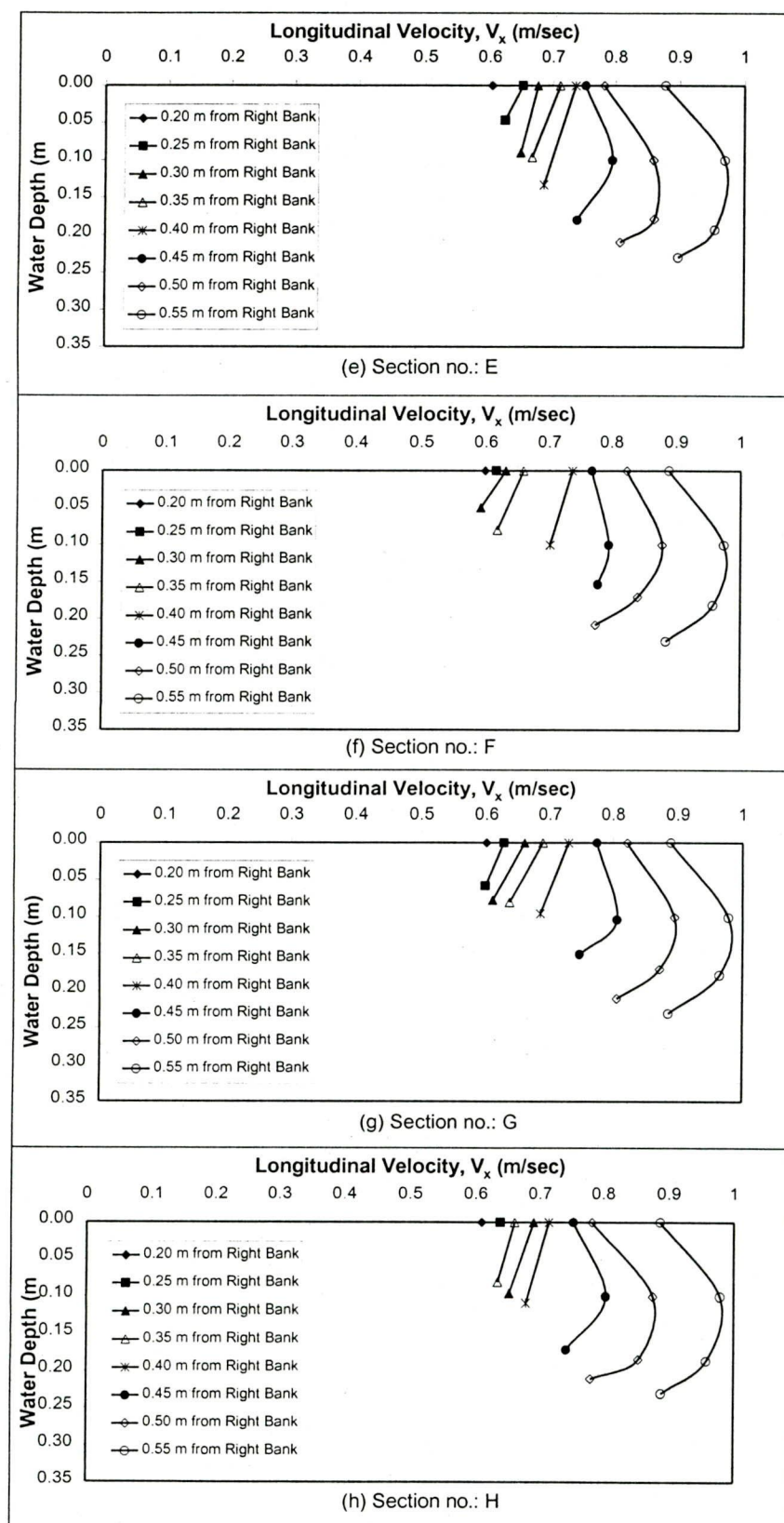


Fig. D.16 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections E to H for cubical type revetments with 3:1 bank slope

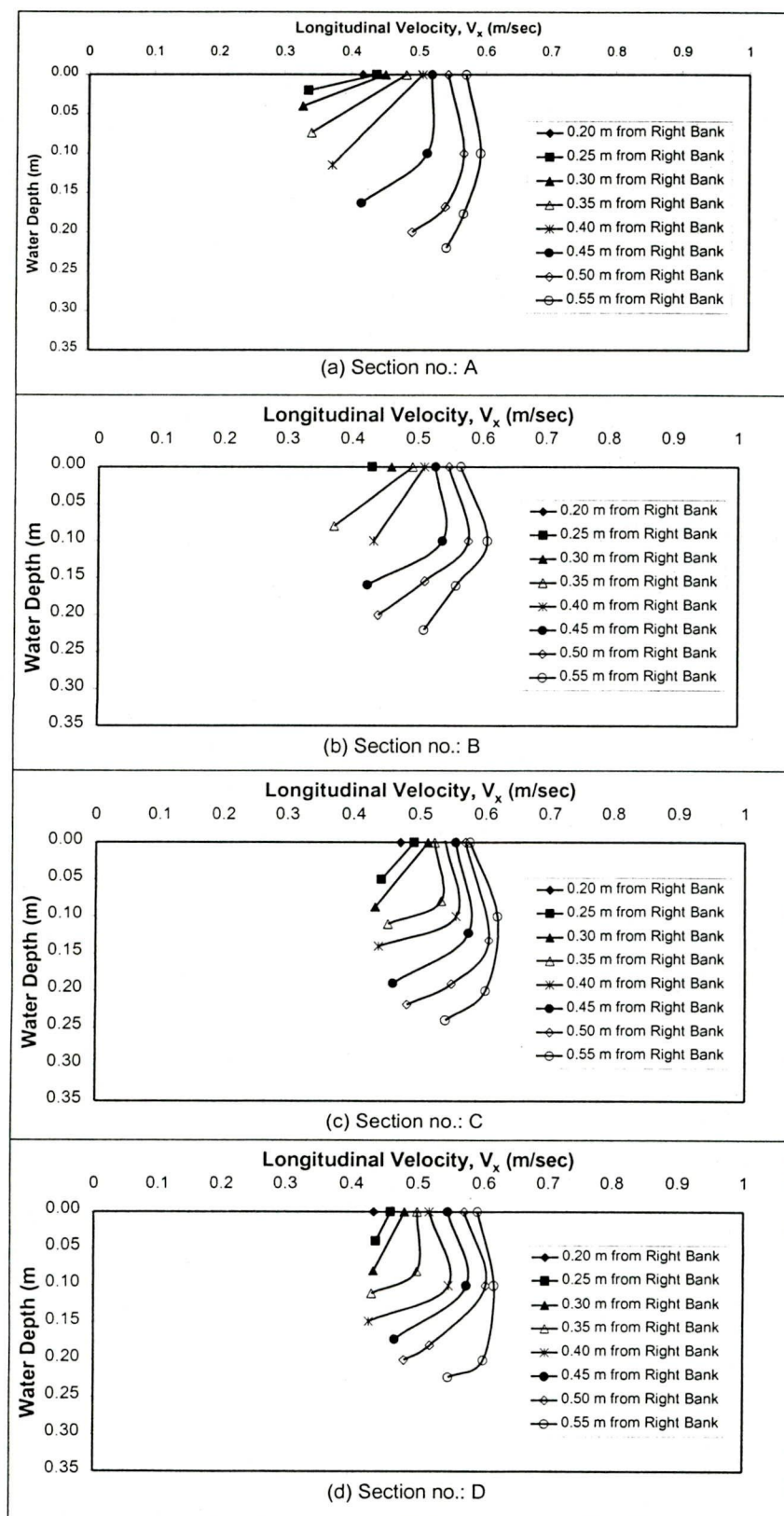


Fig. D.17 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections A to D for hexagonal type revetments with 2:1 bank slope

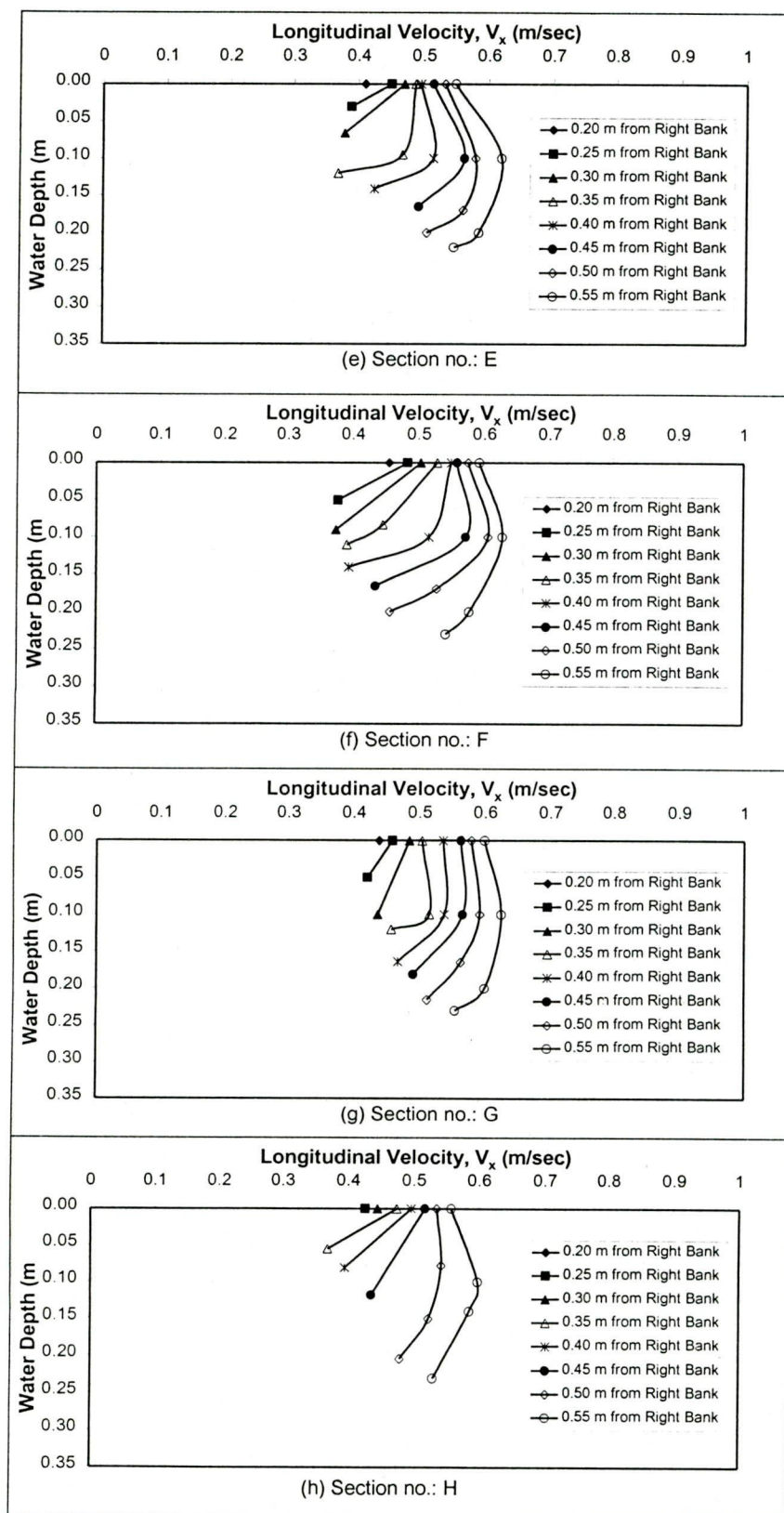


Fig. D.18 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections E to H for hexagonal type revetments with 2:1 bank slope

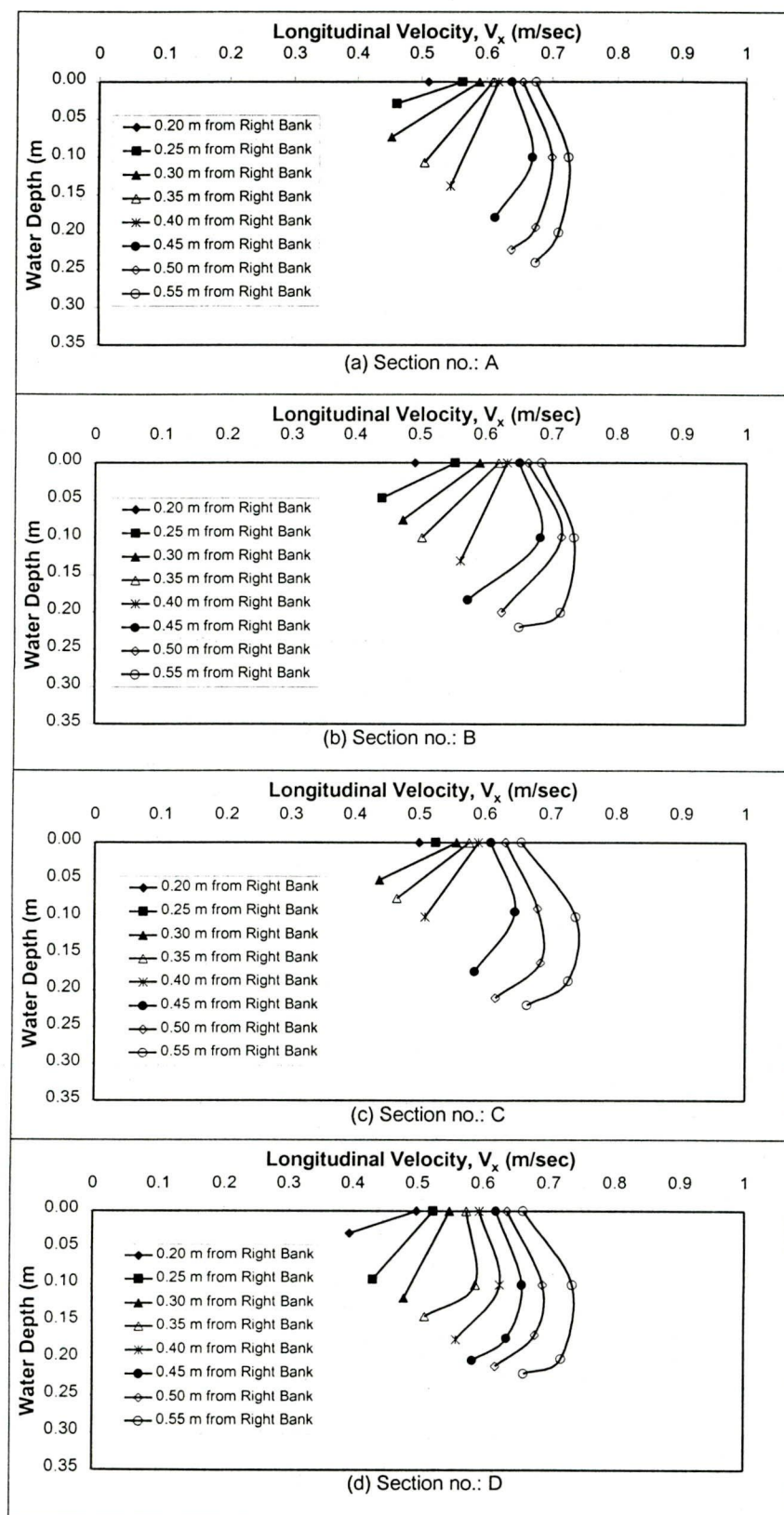


Fig. D.19 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections A to D for hexagonal type revetments with 2:1 bank slope

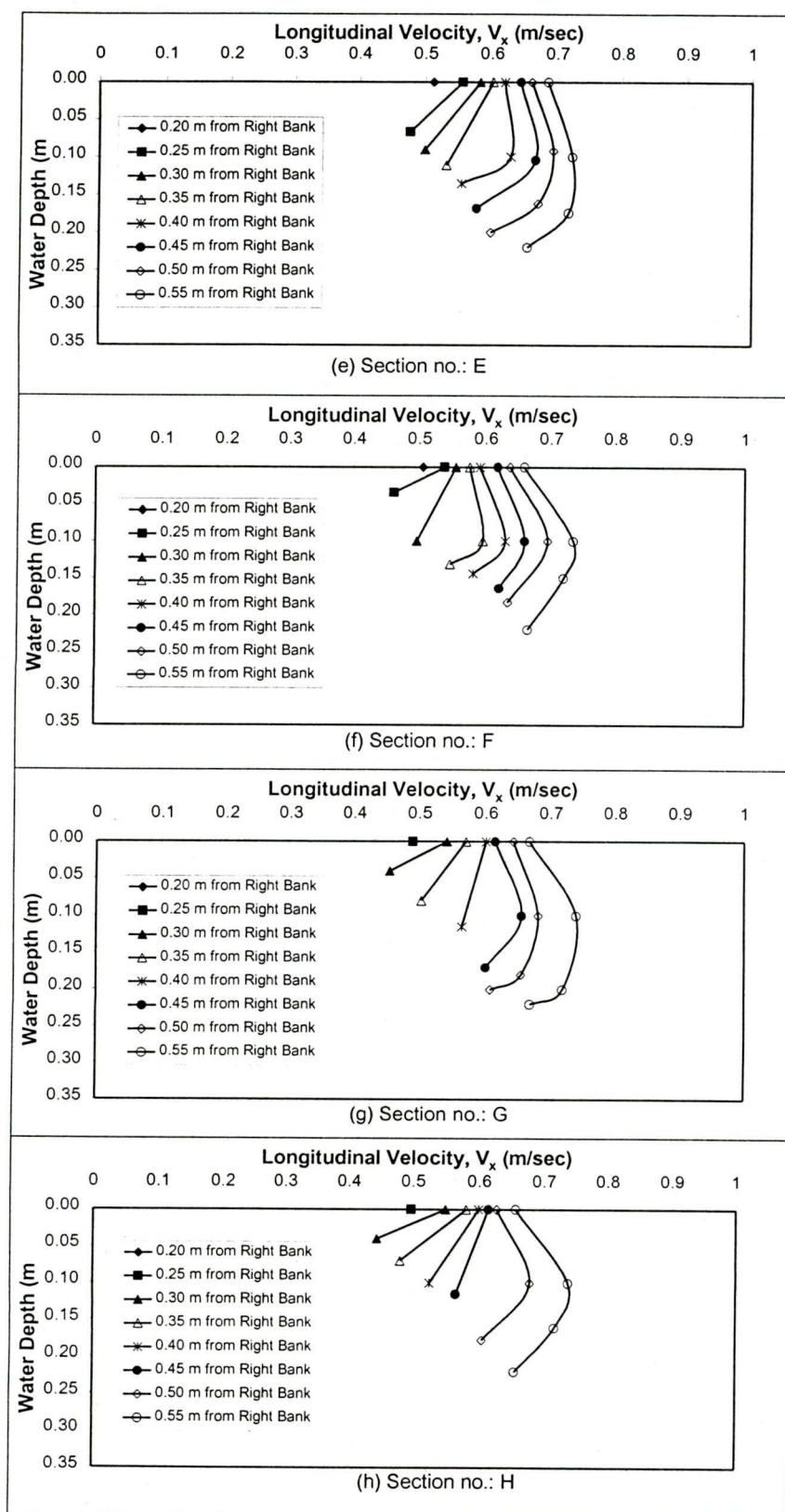


Fig. D.20 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections E to H for hexagonal type revetments with 2:1 bank slope

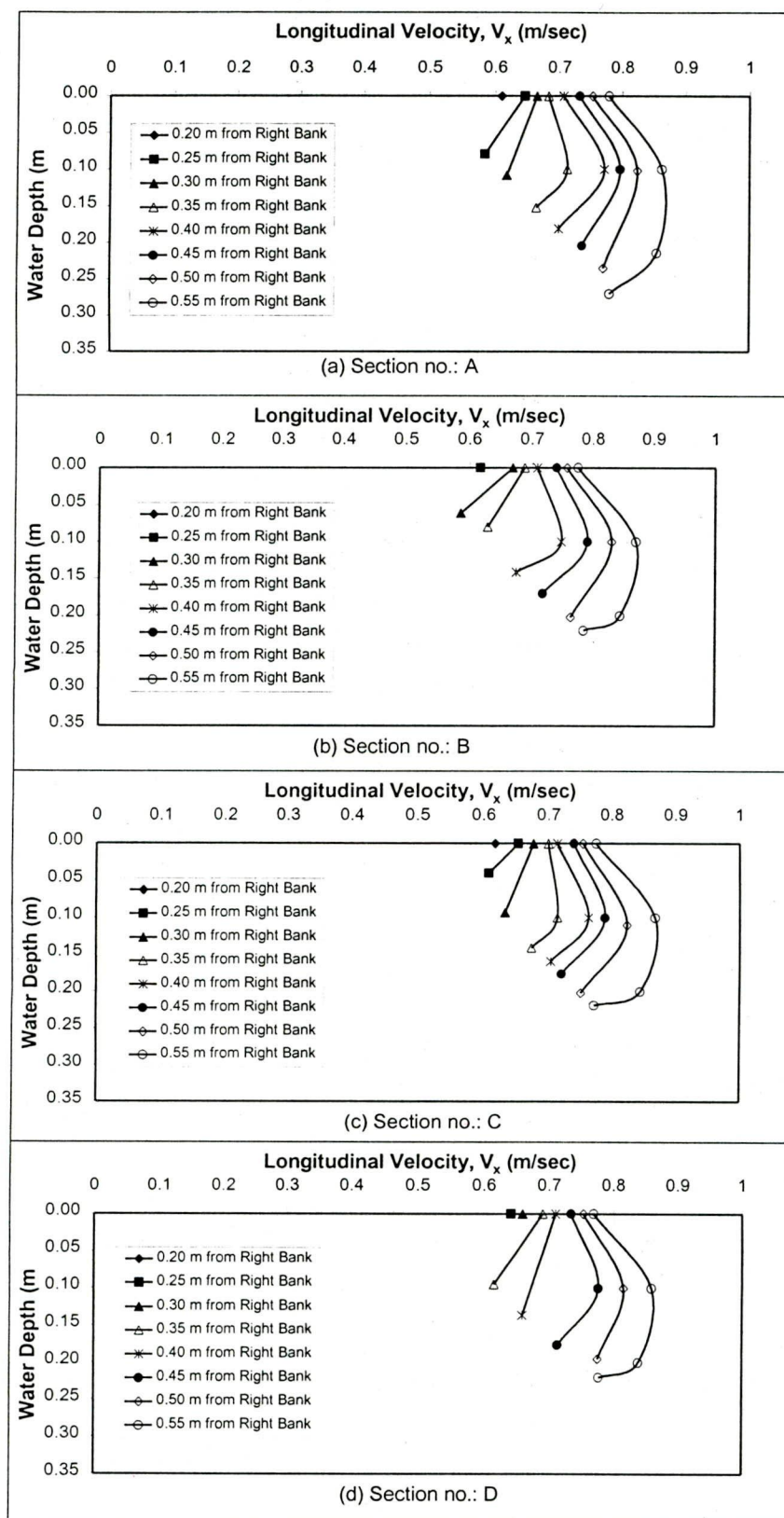


Fig. D.21 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections A to D for hexagonal type revetments with 2:1 bank slope

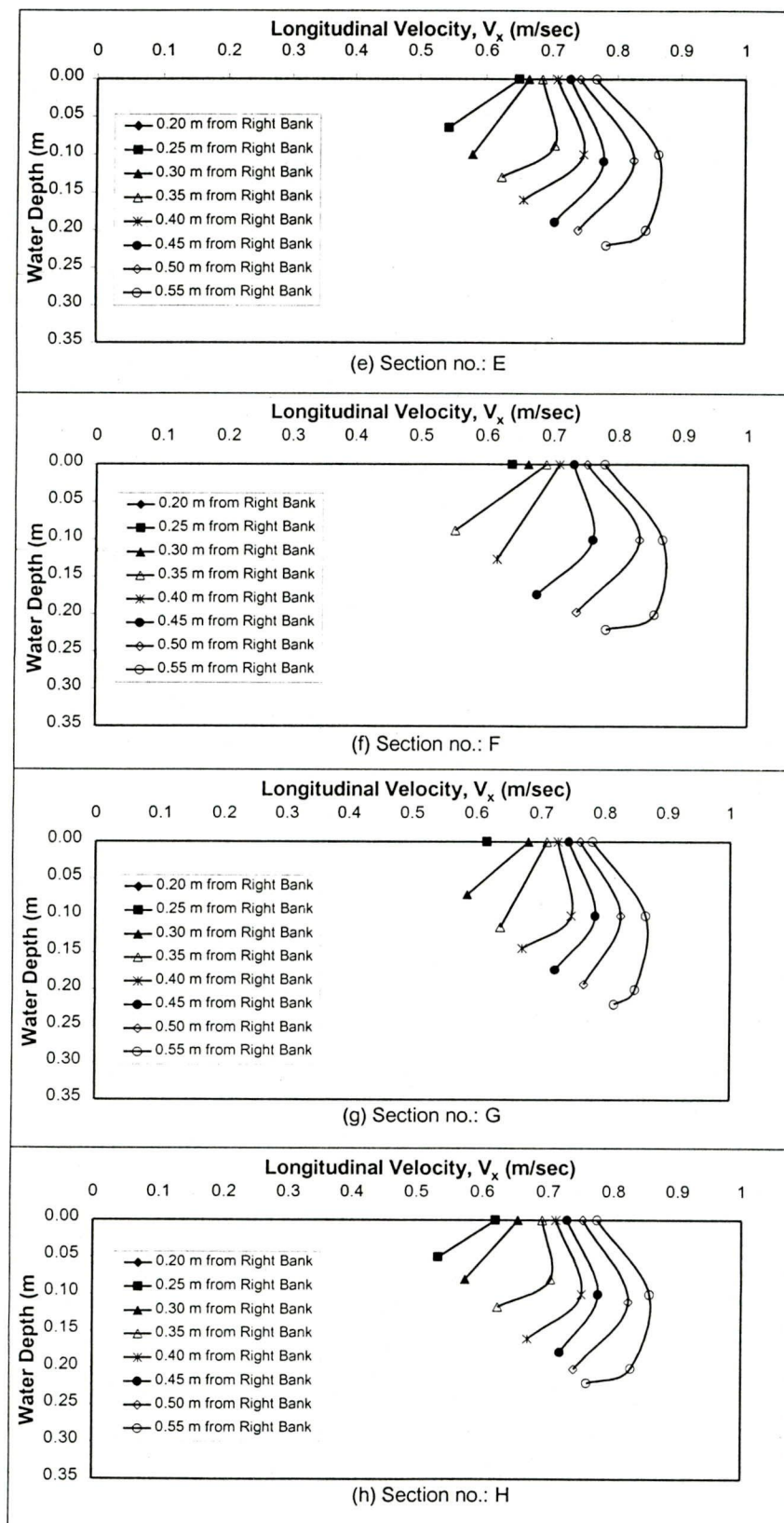


Fig. D.22 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections E to H for hexagonal type revetments with 2:1 bank slope

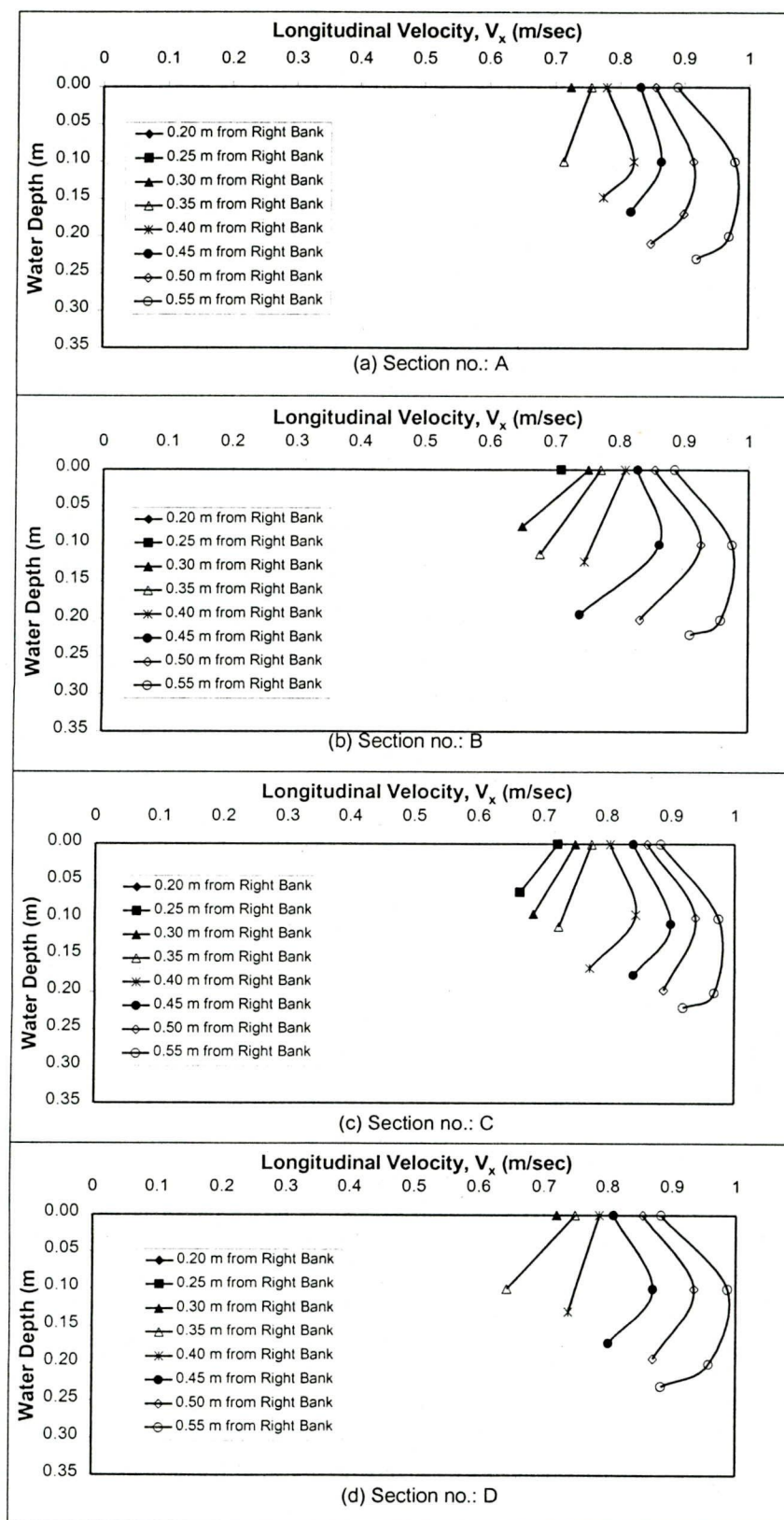


Fig. D.23 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections A to D for hexagonal type revetments with 2:1 bank slope

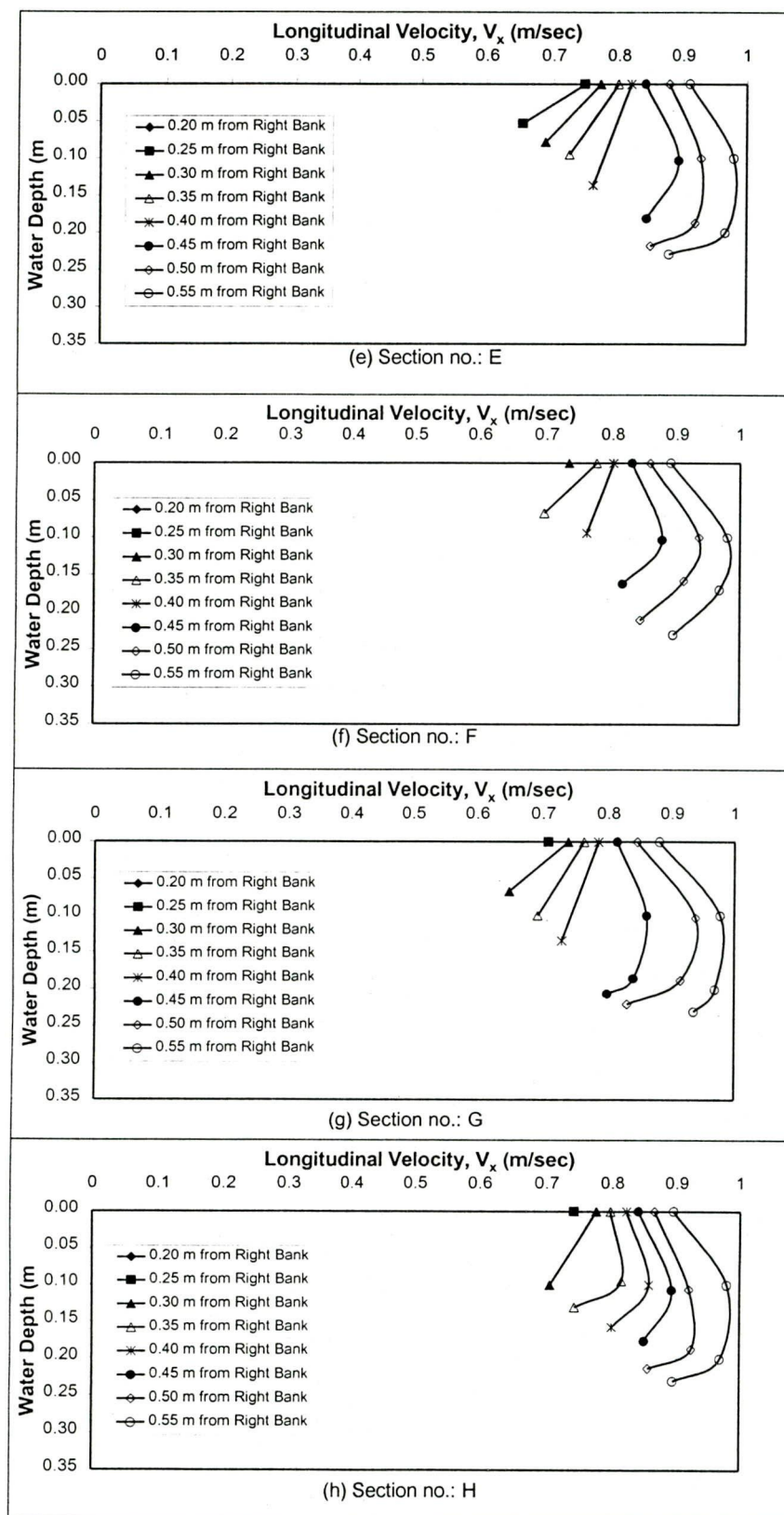


Fig. D.24 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections E to H for hexagonal type revetments with 2:1 bank slope

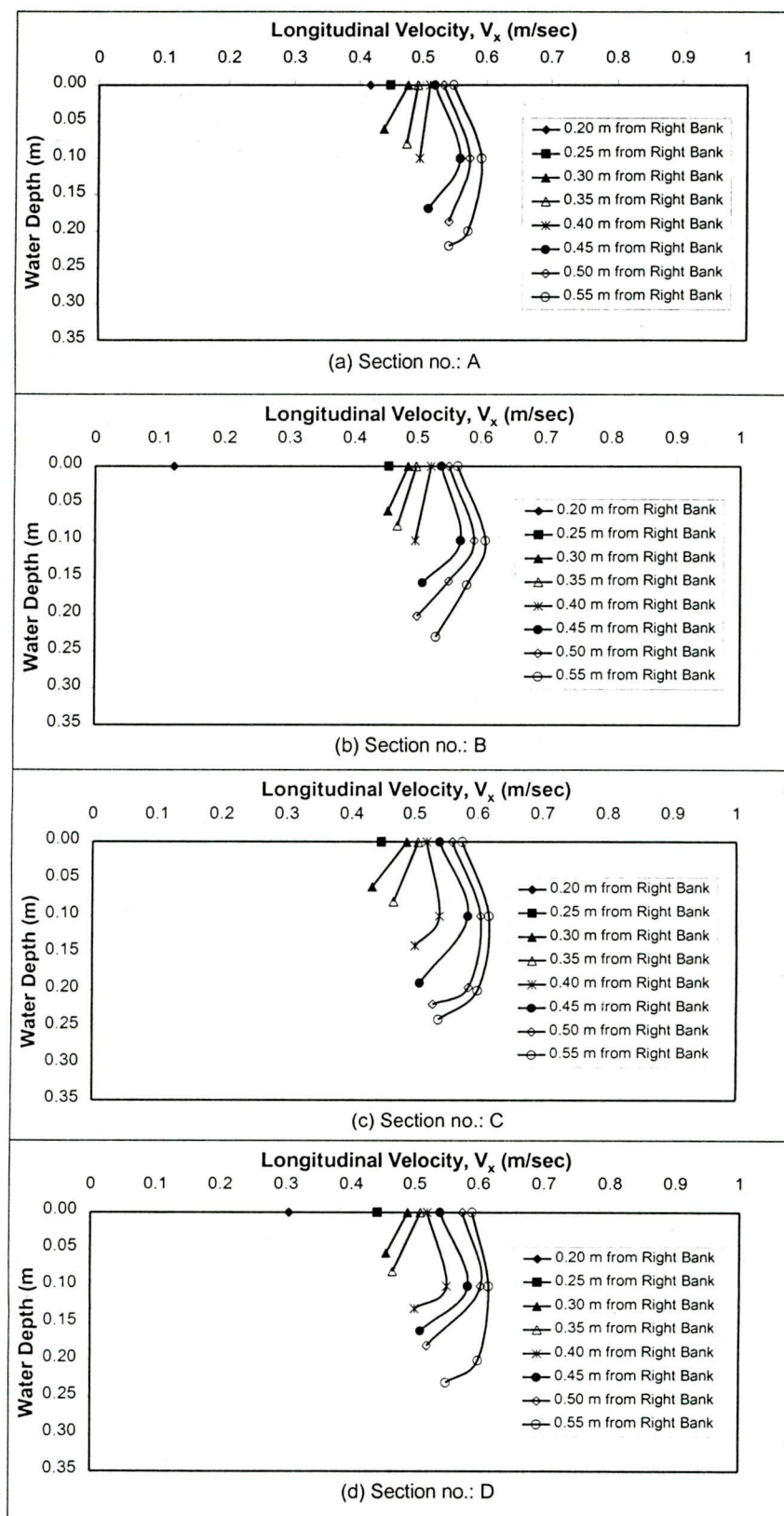


Fig. D.25 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections A to D for hexagonal type revetments with 3:1 bank slope

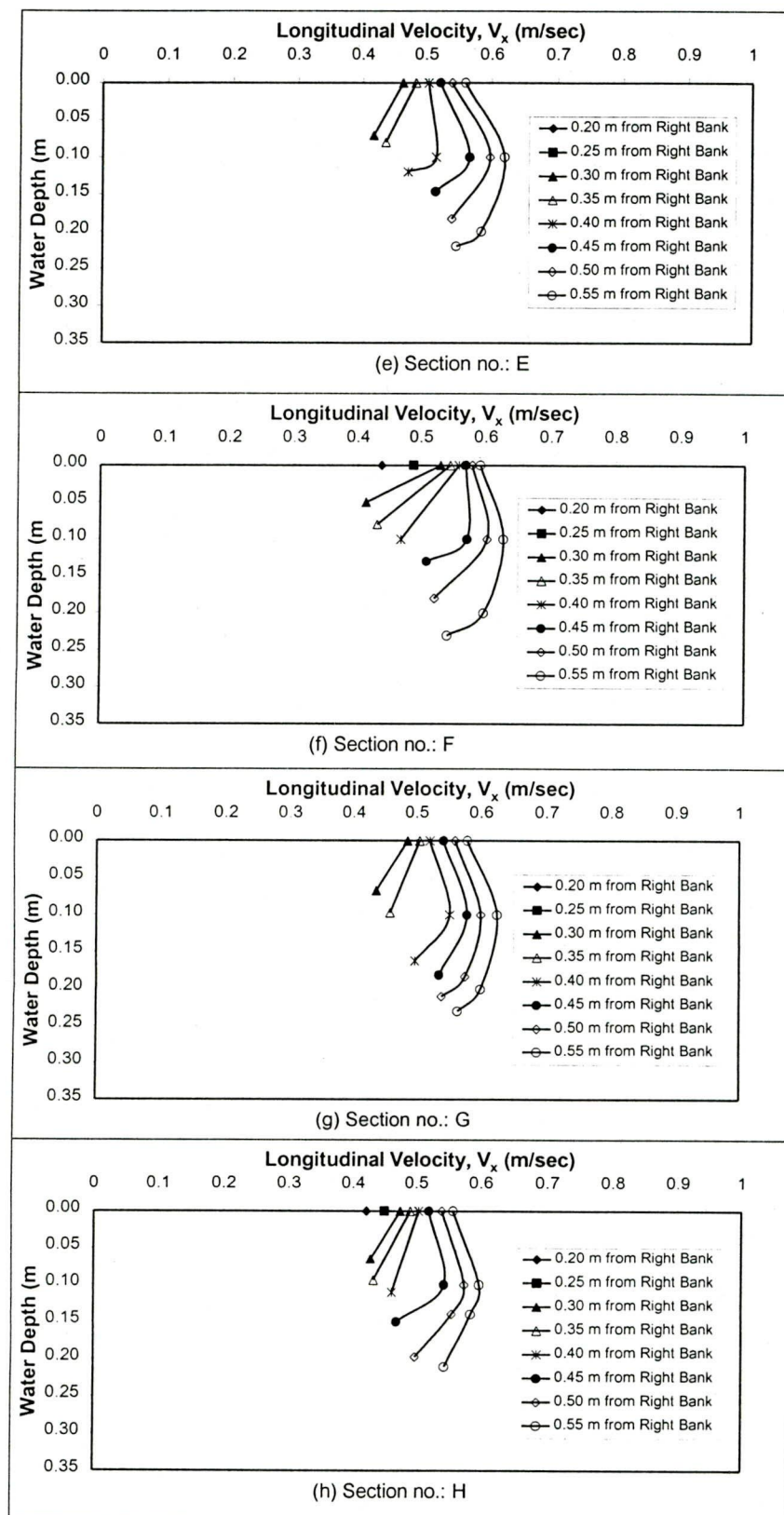


Fig. D.26 Stream wise near bank velocity distribution of the protected study area for average velocity of 50 cm/sec for different sections E to H for hexagonal type revetments with 3:1 bank slope

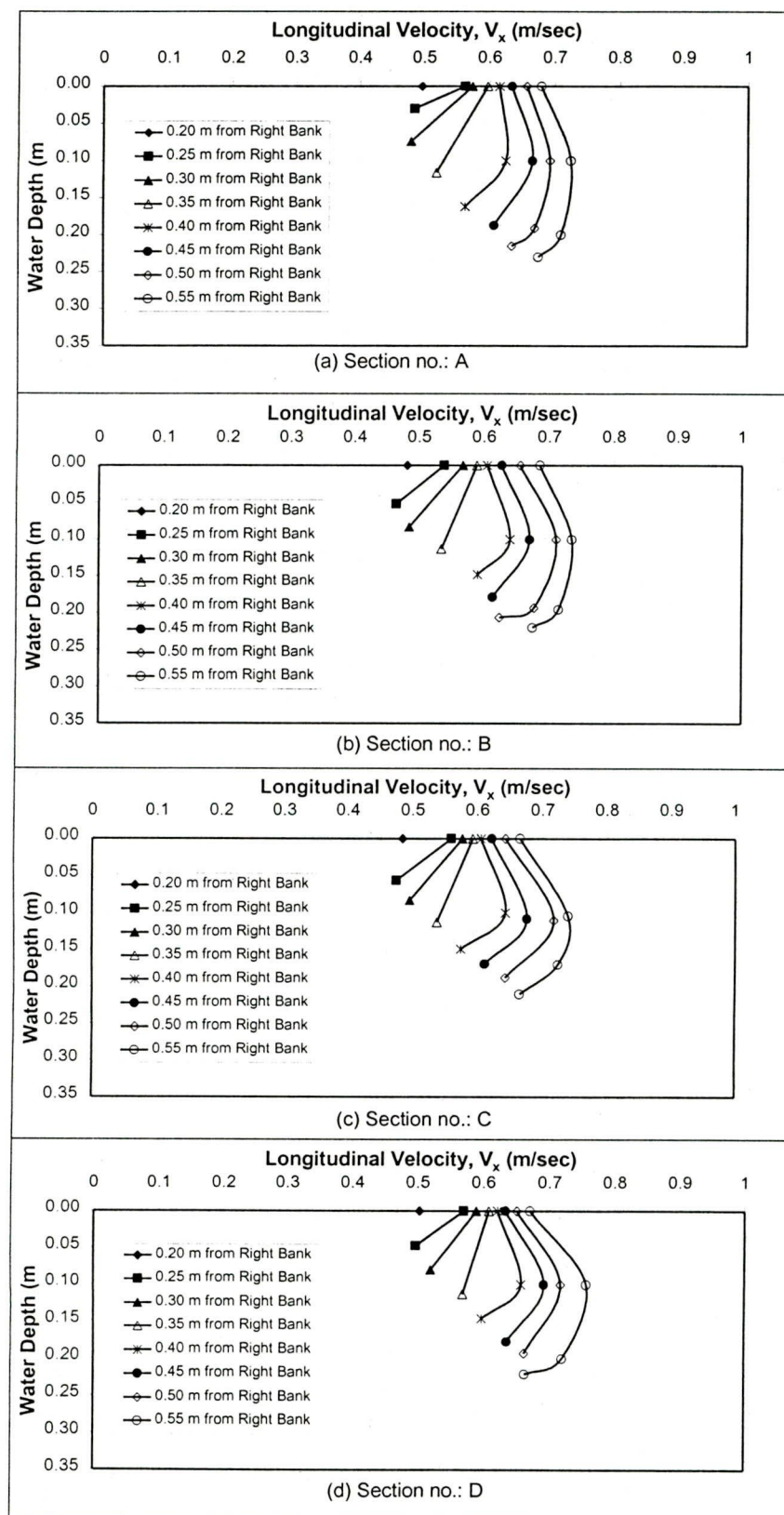


Fig. D.27 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections A to D for hexagonal type revetments with 3:1 bank slope

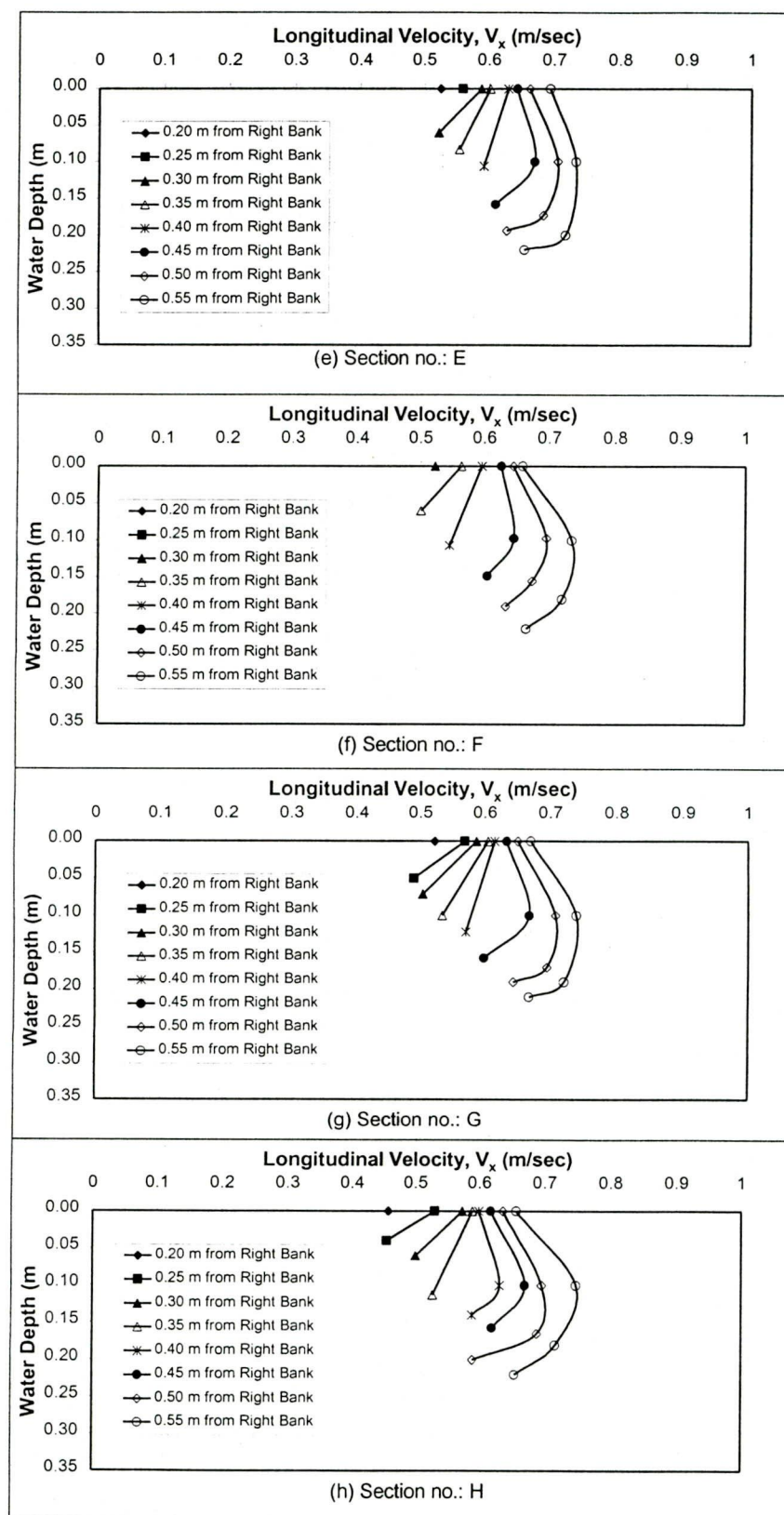


Fig. D.28 Stream wise near bank velocity distribution of the protected study area for average velocity of 60 cm/sec for different sections E to H for hexagonal type revetments with 3:1 bank slope

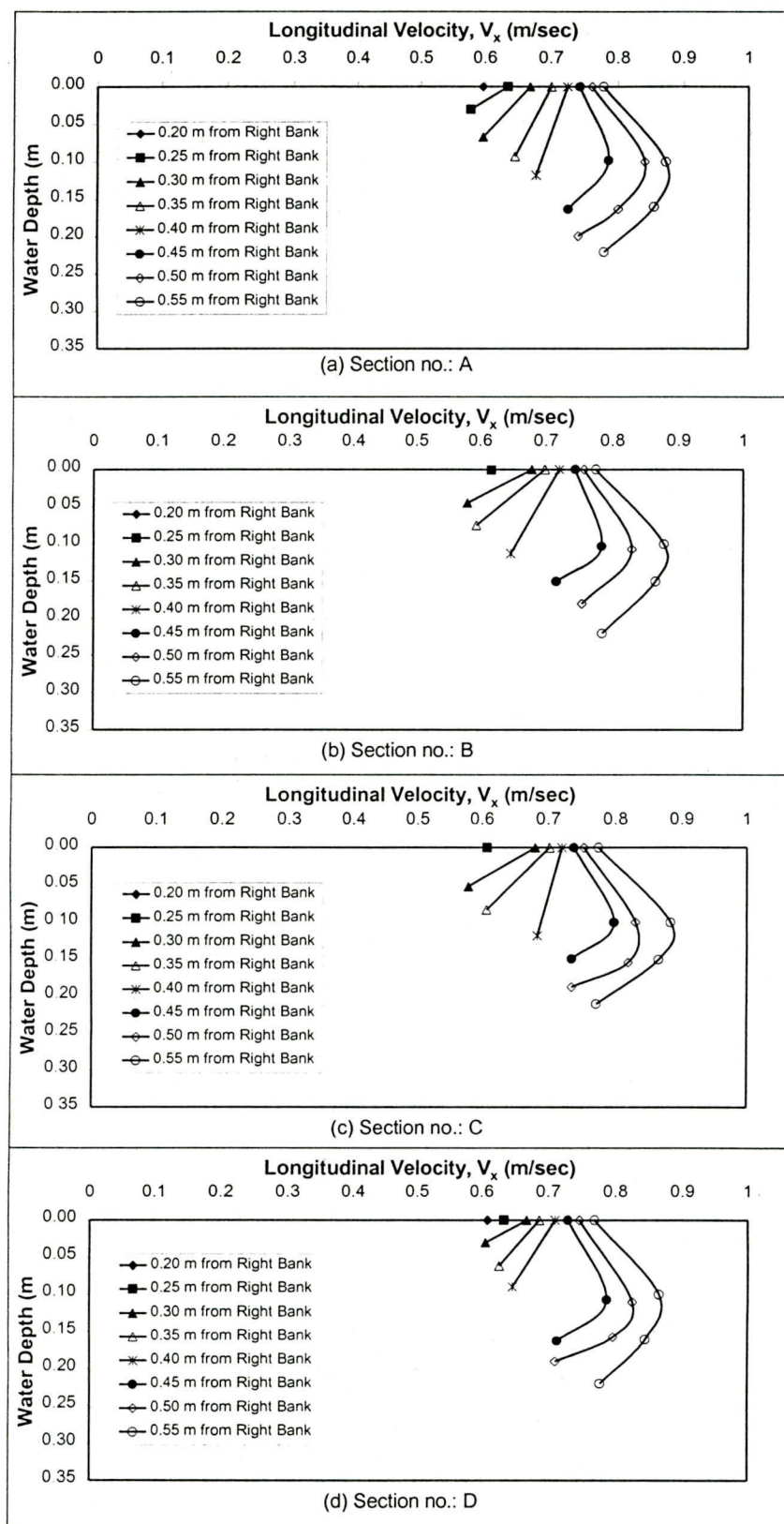


Fig. D.29 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections A to D for hexagonal type revetments with 3:1 bank slope

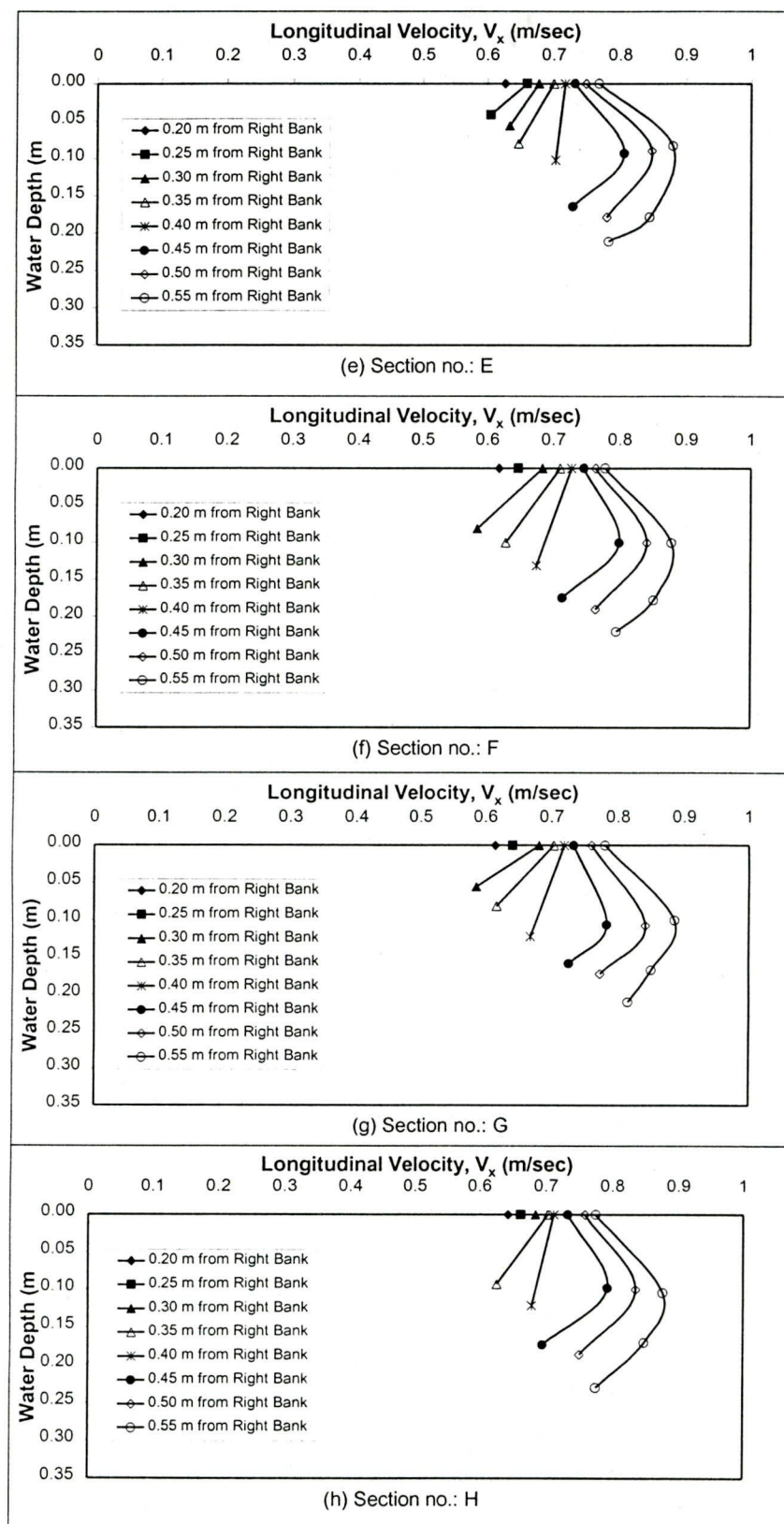


Fig. D.30 Stream wise near bank velocity distribution of the protected study area for average velocity of 70 cm/sec for different sections E to H for hexagonal type revetments with 3:1 bank slope

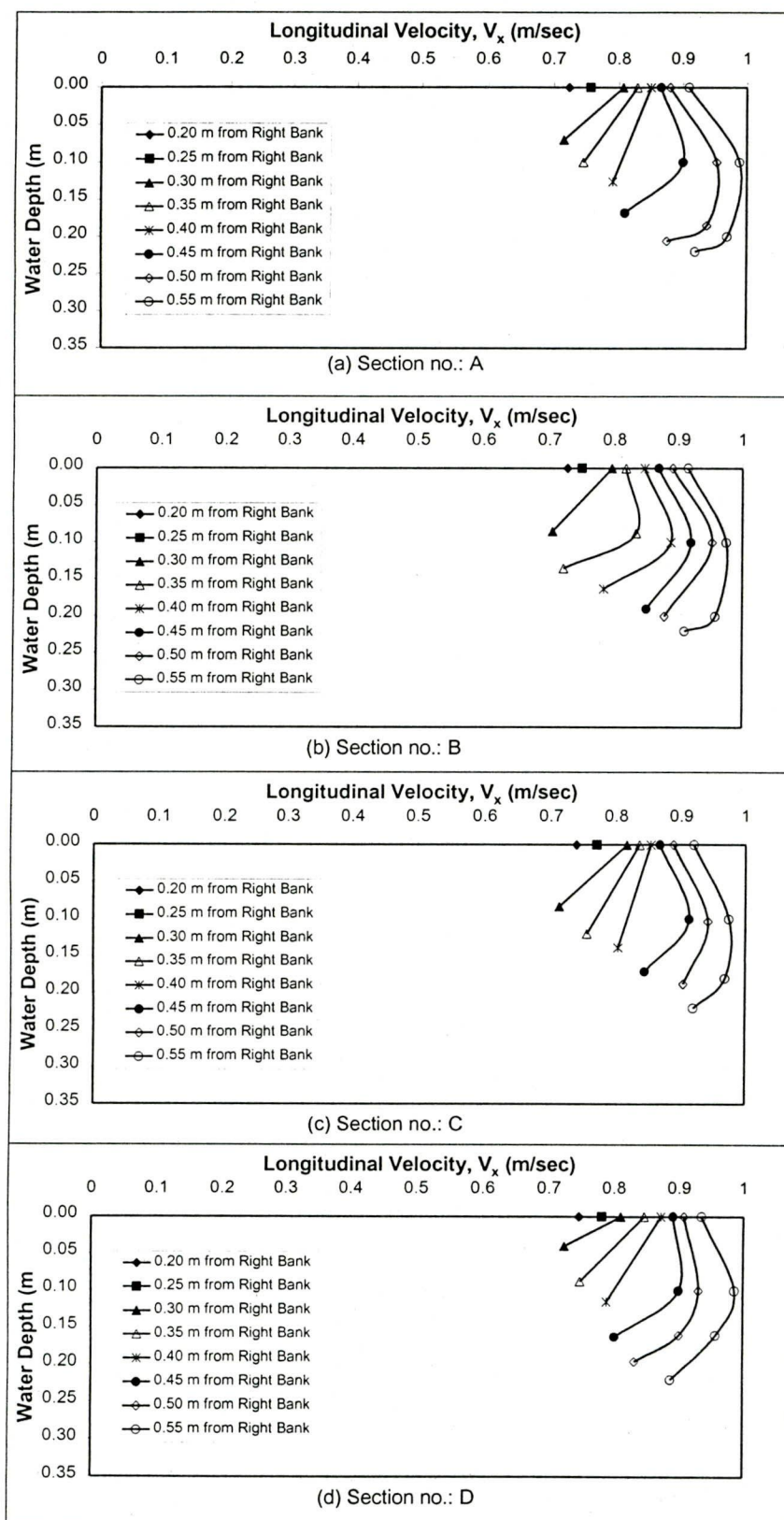


Fig. D.31 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections A to D for hexagonal type revetments with 3:1 bank slope

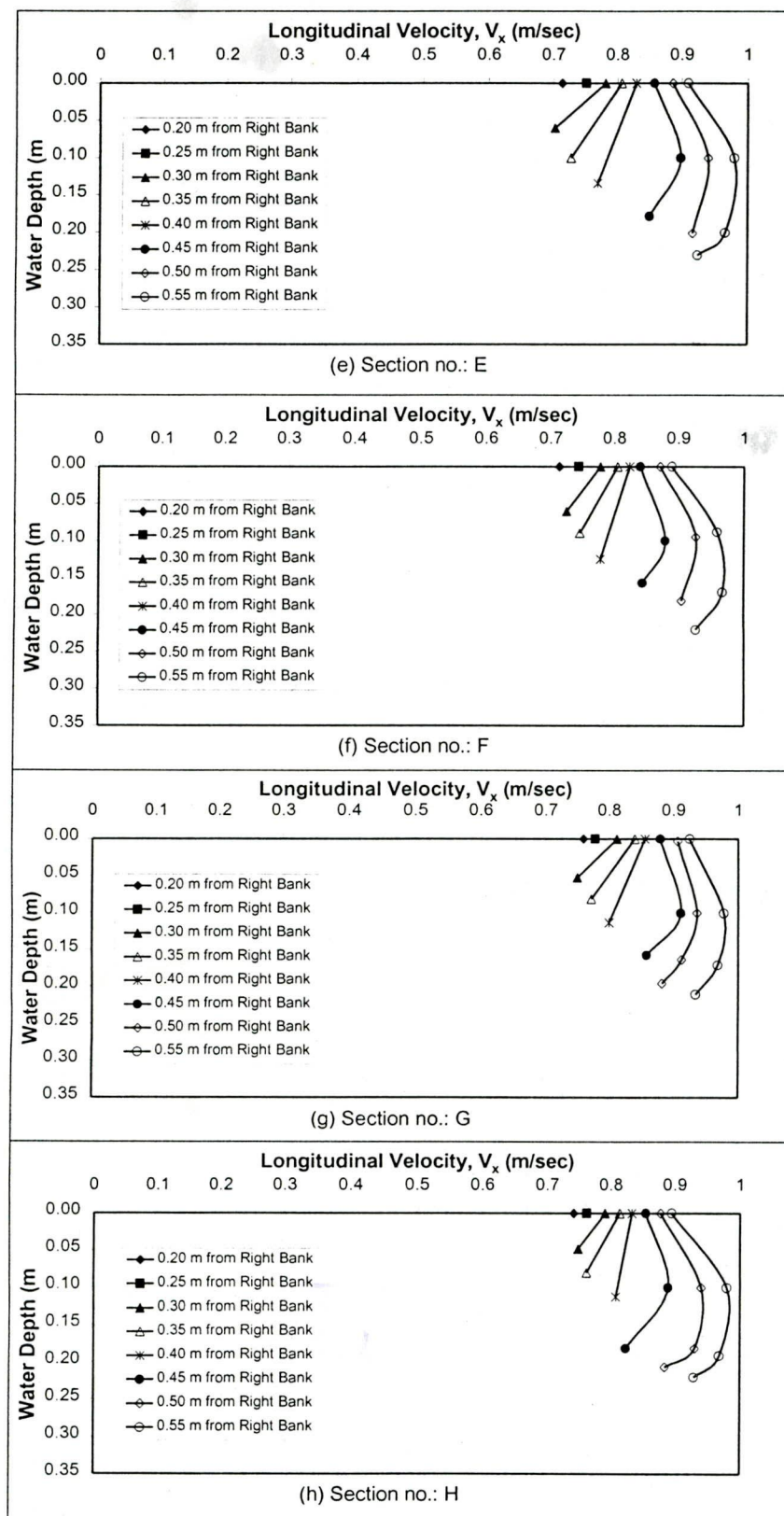


Fig. D.32 Stream wise near bank velocity distribution of the protected study area for average velocity of 80 cm/sec for different sections E to H for hexagonal type revetments with 3:1 bank slope

