DESIGN CIRCLE-II
BWDB DHAKA

BANGLADESH WATER DEVELOPMENT BOARD

GUIDE TO PLANNING AND DESIGN OF RIVER TRAINING AND BANK PROTECTION WORKS

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P R E F A C E

BWDB designs numerous river training and bank protection works every year through different design circles under Chief Engineer, Design-I and Design-II. At present, there are no standard design criteria, procedures and guidelines, based on which different design circles can undertake design of river training and bank protection works. These requirements are met in this report, 'Guide to Planning and Design of River Training and bank Protection Works'. It is expected that various design circles will follow the standard criteria from this guideline.

This guideline incorporate various formulae for design of size of revetment materials currently used by various authors and one or two formulae has been recommended for use. Moreover, the various criteria for planning the different types of river training structures have been elaborated. A typical illustrated example on the design of bank revetment works has also been provided.

This guideline is arranged in eight chapters. The chapter-1 and chapter-2 describe the general aspects of type of river, river training works implemented in Bangladesh, causes of bank recession and purpose of river training. Chapter-3 includes the various types of river training works and their uses. Chapter-4 and chapter-5 describe the hydraulic, geologic, seismic and geotechnical factors need to be considered for design of river training works. Chapter-6 and chapter-7 elaborate the planning and design principles respectively. Chapter-8 illustrates a typical example on bank revetment works.

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Comments and suggestions on materials in this guideline are invite for incorporation in future reviews and amendments.

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Basic Conversion Factors :

The following equivalents of SI units are given in imperial and where applicable, Metric Technical Units.

1 mm = 0.03937 in 1 m = 3.281 ft = 1.094 yd 1 km = 0.6214 mile	1 in = 25.4 mm 1 ft = 0.3048 m 1 yd = 0.9144 m 1 mile = 1.609 km
1 mm^2 = 0.00155 in^2 1 m^2 = 10.76 ft^2 1 m^2 = 1.196 yd^2 1 hectare = 2.471 acres = 10000 m^2	1 in^2 = 645.2 mm^2 1 ft^2 = 0.0929 m^2 1 yd^2 = 0.8361 m^2 1 acre = 0.4047 hectares
1 mm^3 = 0.00006102 in^3 1 m^3 = 35.31 ft^3 = 1.308 yd^3	1 in^3 = 16390 mm^3 1 ft^3 = 0.02832 m^3 1 yd^3 = 0.7646 m^3

Force :

1 N = 0.2248 lb = 0.102 kg 1 kg = 2.205 lb 1 KN = 0.1004 Ton = 102.0 kg = 0.102 Tonne	1 lb = 0.4536 kg 1 Ton = 9.964 KN = 1016 kg = 1.016 Tonne
1 Tonne = 1000 kg = 0.9842 Ton = 9.807 KN	1 Cwt = 112 lbs = 50.80 kg = 0.508 Quintal
1 Quintal = 1.9684 Cwt	

1 Kn = 102.0 Kg

= 224.91 lb = 0.2249

Force Per Unit Length :

1 N/m = 0.06852 lb/ft	1 KN/m = 0.0306 Ton/ft
= 0.1020 kg/m	= 0.1020 Tonne/m
1 lb/ft = 14.59 N/m	1 Ton/ft = 32.69 KN/m
= 1.488 kg/m	= 3.333 Tonne/m

Force Per Unit Area :

1 N/mm^2 = 145.0 lb/in^2	1 Ton/in^2 = 15.44 N/mm
= 10.20 kg/cm^2	= 157.5 kg/cm^2
1 lb/in^2 = 0.006895 N/mm^2	
= 0.0703 kg/cm^2	1 N/mm^2 = 9.324 Ton/ft^2
1 N/m^2 = 0.02089 lb/ft^2	
= 0.102 kg/m^2	

Force Per Unit Volume :

1 N/m^3 = 0.006366 lb/ft^3	1 lb/in^3 = 271.4 KN/m^3
1 lb/ft^3 = 157.1 N/m^3	= 27.68 $Tonne/m^3$
= 16.02 kg/m^3	1 KN/m^3 = 6.366 lb/ft^3
1 KN/m^3 = 0.003684 lb/in^3	1 Ton/ft^3 = 351.9 KN/m^3
= 0.102 $Tonne/m^3$	= 35.88 $Tonne/m^3$

Fluid Capacity :

1 litre = 0.22 Imperial gallons = 0.2642 US gallons
1 Imperial gallon = 4.546 litres = 1.201 US gallons

Power :

1 H.P. = 0.7457 Kilowatts	1 Kilowatt = 1.341 H.P.
	1 Kilowatt-hr = 1.341 H.P.-hr

GUIDE TO PLANNING AND DESIGN OF
RIVER TRAINING AND BANK PROTECTION WORKS

CHAPTER 1

INTRODUCTION

CHAPTER 1

1.0 INTRODUCTION

1.1 General

Rivers have occupied a very prominent place in every stage of human development. Rivers have always been satisfying waterway, domestic, municipal, irrigation and other demands for human being. That is why most of the cities were established in the vicinity of natural rivers. Without any control on these rivers, they cause tremendous devastation and trouble to human beings. With the development of science and technology and in a bid to control nature, man has devised and devising ways and means to control the rivers. In the above context, the study of rivers and their behaviors are necessary to develop ways and means to ensure effective control upon the rivers.

1.2 Types of Rivers and Their Characteristics

Rivers can usually be divided, according to the topography of the river basin, into two parts; the upper reaches in the hilly region and the lower reaches on the alluvial plain. Rivers in the hilly region are characterized by the steepness of the slope, the swiftness of the flow, the occurrence of land slides and the formation of rapids along their courses. The control of rivers in the head reaches is known as "Torrent Control" and the methods adopted are distinctly different from those applied on alluvial rivers known as "River Training".

Alluvial rivers are characterized by the fact that the alluvia, on which the rivers flow, are built up by the rivers themselves. Depending on the material carried by the rivers, the alluvia may be composed of fine silt over large areas; or of shingles and gravels.

Rivers on alluvial plains may be broadly classified into thre types; (a) the meandering type, (b) the aggrading type and (c) th degrading type.

A river on an alluvial plain is seldom of a single type, but al three types may be found on the same river from its uppermost poi on the alluvial plain to its outfall. The classification may sole depend upon the amount and the size of sediment entering the riv and its carrying capacity for the sediment load. As the sedime load and discharge vary with respect to time, any particul section of the river may be aggrading, degrading and meandering different times.

Generally speaking, where a river has enough capacity to carry th incoming sediment downstream without forming large deposits, th whole or a part of it would be of the meandering type.

Owing to excessive sediment entering a river with sudden diminuti of slope on the plain, or owing to the extension of the delta the river outfall, or to sudden inrushing of sediment from tributary, the river will build up its bed to a certain slope a is of an aggrading type.

Degrading type of a river, or a rather section of a river, is fou above a cut-off or below a dam or a barrage, and results eith from the sudden lowering of water surface due to the cut-off whi increases its slope of flow, or from the sudden diminution of i sediment load caused by the damming up of the water above.

Among the three types of rivers, the meandering type is the fu and final stage of development while the other two are inter types which might be maintained as long as the same causes a effects remain unchanged. It will also be seen that the meanderi stage of the river is by no means permanent, as any change in t balance, caused by the natural or artificial form, would affect t river regime. But there is a definite regime for each river follow, with its distance and elevation from the source to the se the magnitude and variation of its discharge and sediment load a the composition of soil composing its bed and banks.

1.3 Rivers of Bangladesh

Bangladesh is primarily a deltaic and a major riverine country. Besides 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 (fig.1.1). These rivers carry 90% of the discharge from beyond the border. About 70 to 80 percent of the annual rainfall in Bangladesh concentrated during the four monsoon months. Runoff from rainfall combined with snowmelt runoff from the Himalayas, causes peak floods which are many times greater than the dry monsoon flood. Along with the flood discharge they also carry an enormous amount of sediment load, estimated at about 2.0 to 4.0 billion tons annually.

1.4 River Training and Bank Protection Works Practiced in Bangladesh

The rivers of Bangladesh continually erode one bank or build the other. The process of erosion has engulfed large areas of cities and towns, agricultural lands and embankments. Erosion specially, by the main rivers, is caused by the inherent character of the alluvial soils and the prevailing monsoon climate.

The large discharge and heavy sediment load cause the rivers to be unstable and the channels are constantly migrating laterally. This instability in river regime coupled with huge discharge and sediment load causes erosion, scouring and also deposition, and a chain action proceeds. It is estimated that about 1200 km. of bank length of rivers are subjected to erosion and of which about 565 km. face severe erosion problem (BWDB, 1984).

Report of BWDB (1984) listed 305 places (565 km.) of river banks and 85 towns, villages which are subjected to severe erosion. It is observed that Brahmaputra-Jamuna is eroding at 41 places, Ganges-Padma at 26 places, Meghna at 6 places, Teesta at 11 places, Kushiyara at 17 places, Khowai at 20 places, Surma at 5 places, Monu at 26 places, Juri at 10 places, Sangu at 7 places, Dhalai at 18 places, and Kankri at 23 places. In addition to those listed above, erosion in small reaches in different places are occurring in almost all the rivers of Bangladesh. Some of the important town where "Town Protection Schemes" have been taken up are Rajshahi o the Ganges. Dinajpur on the Punarbhaba, Serajgonj on the Jamuna Chilmari on the Brahmaputra. Bhairab bazar on the Meghna, Chandpu on the Meghna & the Dakatia, Khulna on the Bhairab & the Rupsha Rangamati on Kaptai Lake etc. The summary of river bank erosion ar presented in Table 1.1

TABLE - 1.1

SUMMARY OF RIVER BANK EROSION IN BANGLADESH

Name of River	No. of Location of bank erosion	Length of Erosion (k
a. Brahmaputra-Jamuna	41	162.60
b. Ganges-Padma	26	94.50
c. Meghna	8	72.00
d. Teesta	11	34.90
e. Minor rivers	112	92.30
f. Flashy rivers	75	23.00
g. Tidal rivers	32	85.80
	305	565.10
h. Cities	85	

Ref : Alam S.M. Zakiul & Fraque H.S. Mozadded, "Bank Protection Methods Used in Bangladesh".

Table 1.2 presents different types of River Training Works used in Bangladesh to protect important towns from river bank erosion.

TABLE - 1.2

RIVER TRAINING WORKS USED IN BANGLADESH

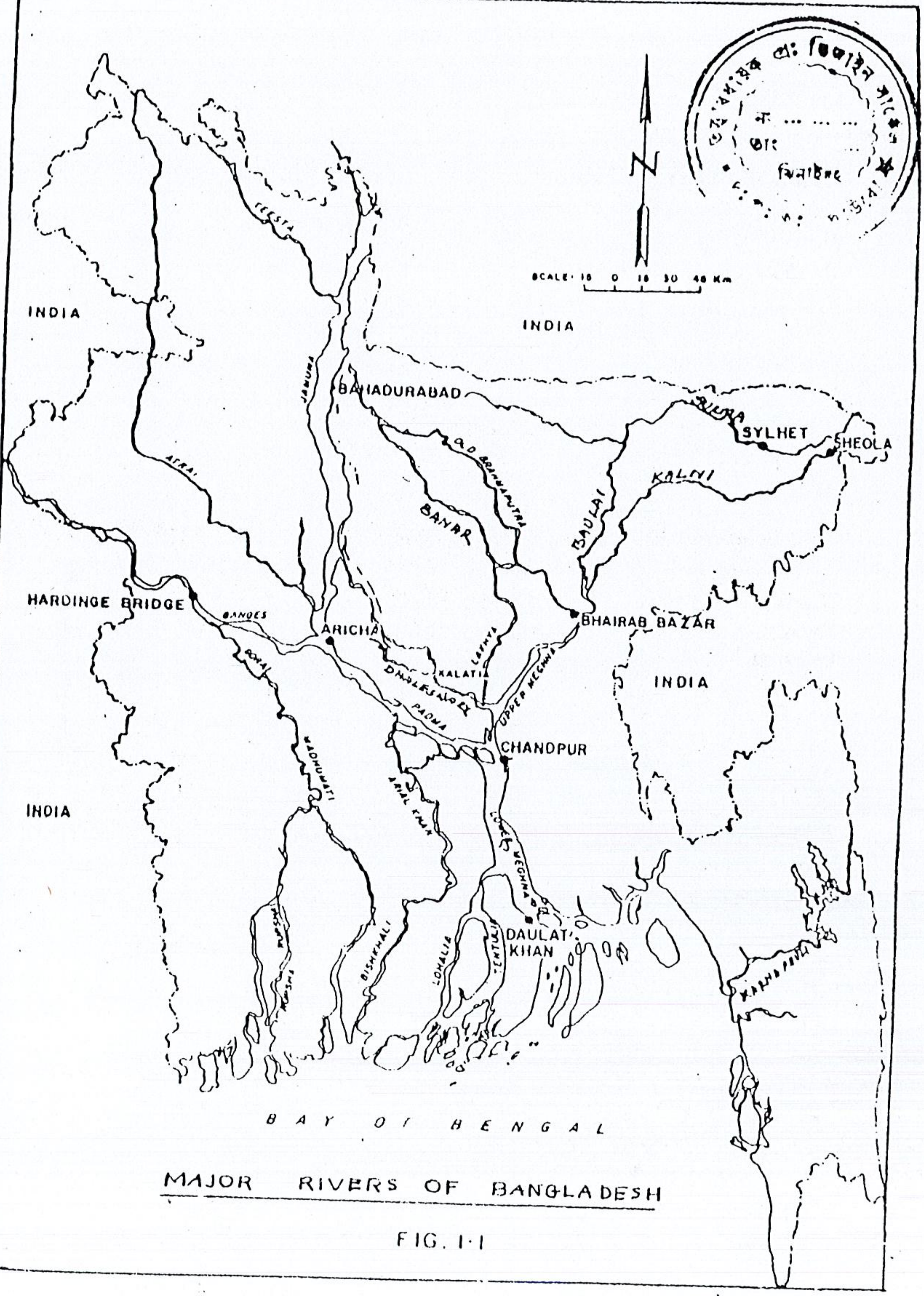
Sl.No.	Name of Scheme	Type of protective Works
1.	Rajshahi Town Protection Scheme	a. Groyne b. Spur c. Brick Matressing
2.	Serajgonj Town Protection Scheme	a. Bank Revetment b. Groyne
3.	Khulna Town Protection Scheme	a. Bank Revetment
4.	Kurigram Town Protection Scheme	a. Groyne b. Bank Revetment c.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

Sl.No.	Name of Scheme	Type of protective Works
9.	Dewanganj Town Protection Scheme	a. Loop Cutting b. Closure Dam
10.	Protection of Bhairab Bazar Town	a. Bank Revetment
11.	Madaripur Town Protection Scheme	a. Bank Revetment
12.	Rangamati Town Protection Scheme	a. Saddle Dam b. Toe wall c. Slope protection d. Revetment on berm e. Retaining wall
13.	Protection of Teesta Banks	a. Groyne b. Bank Revetment
14.	Protection of Brahmaputra Right Flood Embankment	a. Groyne b. Cross Dam
15.	Erosion in Tidal Areas	a. Bank Revetment b. Porcupines
16.	Protection of Zakiganj Town	a. Groyne b. Bank Revetment
17.	Stabilising the right bank of Muhuri River from Bop at Nizkalikapur to Fakirkhil village	a. Bank Revetment b. Spur
18.	River Training works of Feni river at Lacharipara	a. Groyne b. Bank Revetment

Sl.No.	Name of Scheme	Type of protective Works
19.	Bank protection works of Piyain river at Sangrampunji, Nakshipunji in Thana Gowainghat Dist. Sylhet	a. Bank Revetment
20.	River Training works of Matamuhuri at Cheringa	a. Series of guide banks and Spurs b. Improvement of channel capacity c. Flood Embankment
21.	River Training works of Sangu river at Dohazari bridge	a. Groyne b. Bank Revetment

1.6 Scope of the Guidelines

River training implies any construction provided on a river to direct and guide the river flow including floods, to train and regulate the river bed or to increase the low water depth. The objective and scope of the guide line is to provide various methods of river training works in practice with special reference to their applicability in different regions of Bangladesh and the design principles, so that these can be need elsewhere in the country.



MAJOR RIVERS OF BANGLADESH

FIG. 1·1

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CHAPTER 2

CAUSES OF BANK RECESSION AND PURPOSE OF RIVER TRAINING

C H A P T E R 2

2.0 CAUSES OF BANK RECESSION AND PURPOSE OF RIVER TRAINING

2.1 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 slope 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 receeding 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.

The various hydraulic action responsible for erosion in particular location can usually be classified as follows (after California Highway Practice,1960), (fig.2.1).

(a) Frictional erosion by tangential flow.

(b) Impingement erosion by curvilinear flow.

(c) Eddy erosion below restriction.

(d) Kolk scour below reef.

(e) Erosion in varied flow.

(f) Erosion in unsteady flow.

(g) Wave action.

(h) Static erosion.

2.2 Purpose and Objective of River Training Works

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

(a) Safe and expeditious passage of flood flow.

(b) Efficient transportation of suspended and bed load.

(c) Stable river course with minimum bank erosion.

(d) Sufficient depth and good course for navigation.

(e) Direction of flow through a certain defined stretch of the river.

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CHAPTER 3

TYPES OF RIVER TRAINING WORKS

FIG. 2·1

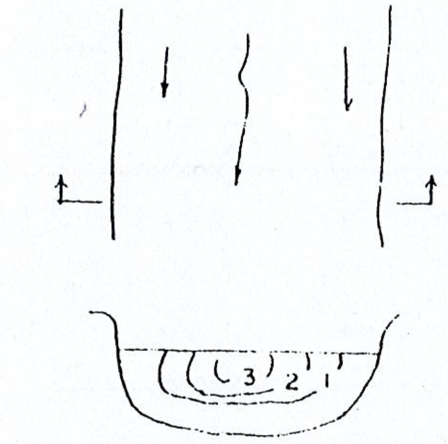


FIG. 1a VELOCITY DISTRIBUTION IN TANGENTIAL FLOW

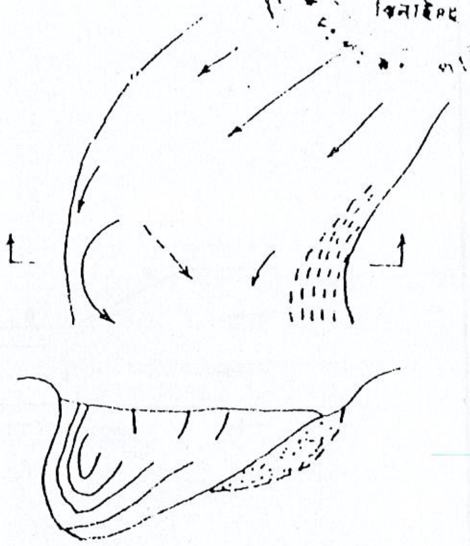


FIG. 1b VELOCITY DISTRIBUTION IN CURVE FLOW

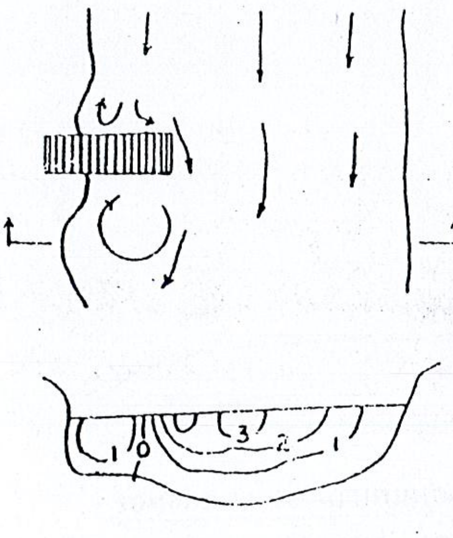


FIG. 1c TURBULENCE IN RESTRICTED CHANNEL

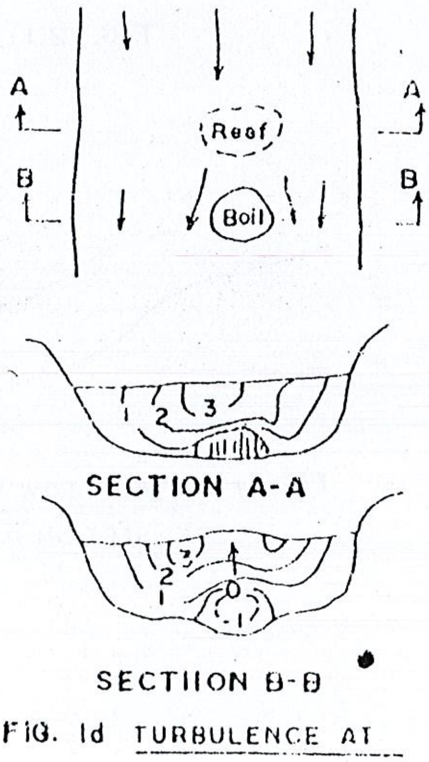


FIG. 1d TURBULENCE AT REEF IN CHANNEL

Source: Nishat A., Control of Erosion and Deposition in river.

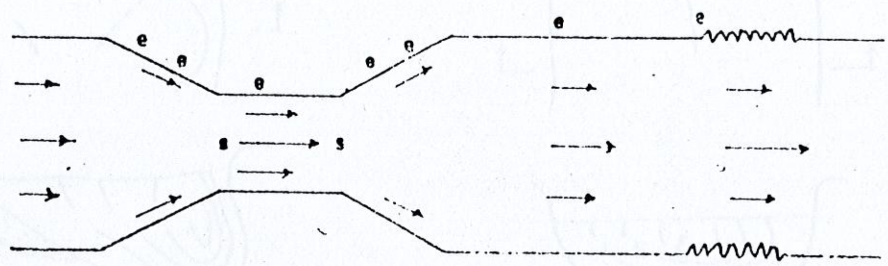


FIG. 2.1

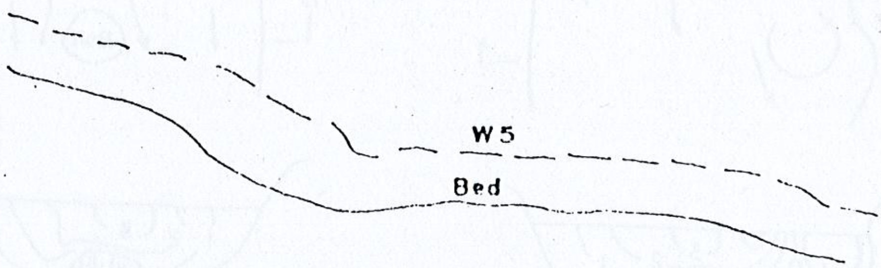


FIG. 1e VARIED FLOW (STEADY NONUNIFORM) AT CHANGE OF SECTION ROUGHNESS OR SHAPE

CHAPTER 3

3.0 TYPES OF RIVER TRAINING WORKS

Type of works to be done for the purpose of training a river or a reach thereof 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 described below with special reference to their important features and applicability.

3.1 Bank Revetment

This type of River training works involve a protective cover of a suitable hard material applied on the slope and toe of the river bank so that the bank soil is protected from the actions of erosive forces of flowing water and dynamic actions of waves. Revetments are of two types :

(a) Open joint type in which joint gaps between individual hard material remains open allowing free flow of water. Examples of open joint revetments are (fig. 3.1)

- Dumped riprap (Stones, boulders, blocks, tetrapods etc.)
- Hand placed riprap with joints kept open (stone, boulders, blocks etc.)
- Gabions and Matresses filled with boulders, stones, blocks, bricks etc.
- Fabric Matresses and Tubes filled with sand, gravel and concrete

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

(b) Close-joint type in which joint gaps between individual hard material are sealed using cement, bitumen or asphaltic material. Close-joint type revetments do not allow free flow of water and are liable to develop detrimental head of water behind which may eventually lead to blow-offs. Examples of close-joint type revetments are (fig. 3.2)

- concrete paving,
- bitumen or asphaltic concrete paving,
- wet (with mortar) pitching (boulders, bricks),
- hand placed riprap (boulders, blocks) with mortared joints.

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

3.2 Groyne

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

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

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

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

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

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

b. Based on materials and method of construction, spurs/groynes are classified as:

. Permeable spurs, which are of porous construction and allow significant part of impinging flow to pass through the body of the structure. Permeable spurs are usually constructed using piles (bullah, bamboo, concrete). In order to be effectively permeable, porosity of the structure may be not less than 40-50 percent. Permeable spurs are suitable where mild or partial deflection of flow is desired. They can, under favourable condition, induce satisfactory sediment deposits in the downstream area. Permeable spurs, made of bullah or bamboo piles should be used in less severe conditions and where short term objectives are to be met because bullah and bamboo piles are short lived and offer weaker construction. However strong and durable permeable spurs can be constructed using concrete (precast or in-situ) piles, one or more rows or clamps of piles or jacks.

. Impermeable spurs/groynes are solid constructions and act as barriers to impinging flow. Impermeable spurs/groynes usually have armoured earthen construction, however, boulders, stones, gabions etc may also be used, if otherwise found feasible. Impermeable groynes are enormous structures and therefore are expensive due mainly to high cost of armouring the earthen core.

c. Depending on shape, spurs/groynes are named as (fig. 3.3)

. Straight spurs/groynes are simple structures projecting straight into the river from the bank. Usually the nose part of a straight groyne needs a little enlarging to accomodate the required quantity of armouring material. Permeable spurs are made, by practice, straight.

- T-head groynes resembles in letter "T", with the T-head positioned in river along the direction of flow. The specially shaped groyne developed in India provides a bett and longer guidance to the flow with its long T-head. The groynes are usually impermeable and many of them have be constructed with varying success.

- Hockey groynes having resemblance to "Hockey Stisks" are also special shaped groynes having special applications. Hockey groynes may be of both permeabile and impermeable type.

d. From functional view points, Spurs/Groynes are classified a (fig. 3.4)

- Attracting type, where due to the intervention by the spur/groyne, incident flow of the river is attracted towar the bank. Attracting spurs are aligned at downstream faci directions. This type of spurs/groynes are useful when t deep channel is required to be along and near a particul bank reach and permeable type of spurs can be attracting aligned.

- Repelling type, where the spurs repel the incident flow the river in a desired direction away from the near ban Repelling groynes are the most aggressive of all the typ and, therefore, need to be relatively long and heavily strengthened. These types of groynes are aligned at upstre facing directions.

- Deflecting type, where the spurs, out of their interventio change the direction of incident flow with a relatively local effect and do not totally repel it away. These types spurs are also aligned at upstream facing directions. Deflecting spurs/groynes are moderately aggressive and, accordingly, their lengths are relatively small. These typ of groynes have been widely used in indirect protective measures against bank recessive forces.

3.3 Guide Bundh

Guide bundhs are artificial banks constructed along planned alignments with the purpose of guiding river flow in a desired direction, course or pattern at structures like bridges, weirs, barrage etc. across the rivers. Basic objectives behind obtaining guided flow at such structures are:

(a) to avoid risk of outflanking,

(b) to prevent erosion of approach roads,

(c) to avoid problems related to obliquity of flow through the structure,

(d) to induce uniform distribution of discharge throughout the waterway.

Guide banks are usually earth embankments with necessary armouring. Generally, guide banks are used in pairs, because structures requiring guide banks are, in most situations, constructed at the middle of wide flood plains or wide braided channels. However, single guide bank in one side only can also be used when a naturally advantageous condition exist on the other side. Guide banks are classified according to their planforms and layout geometry as outlined below (for definition sketches, see Fig. 3.5

A. Parallel Guide Banks

Based on plan form, guide banks are called parallel when the pair of guide banks are laid with their shanks parallel. This type of guide bank have the advantage that;

(a) Flow between the guide lines remains relatively uniform throughout the length of guide bank.

(b) Possibility of formations of shoals infront of the structure is reduced.

(c) Smaller length of guide banks is required to contain embayment upto a desired extent in respect of protection of approach road.

However, parallel guide banks are not suitable in situation involving flows which may approach obliguily.

B. Divergent Guide Banks

Divergent when distance between the two guide banks graduallì increases as on moves from the structure towards upstream Divergent guide banks are advantageous in dealing with flows having oblique approaches. However, they have the disadvantages that shoals are likely to be developed in between the guide banks near the upstream curved heads and they need greater lengths compared to parallel guide banks to contain embayments upto the same extent. Though the above two types of guide banks are generally used, other types of plan forms may also be used if necessity warrents.

While shanks of parallel guide banks are always straight geometrically, shanks of diverging guide banks can be either straight or have elliptical curvature. However, terminations at head and tail of guide banks are formed by either multi radii curves. Heads of guide banks have to withstand severe erosive and scouring actions of flowing water and therefore should be adequately armoured.

3.4 Artificial Loop Cut

Loop-cut (or cut-offs) is a natural process in which an alluvial meandering channel leave the original course around the loop, when the loop length increases much in the shape of horse-shoe and flows relatively straight by cutting off through the narrow neck of the loop. A chute cutoff results generally when the slope becomes inadequate to support the flow and the sediment discharge. Accordingly, loop cuts are sometimes made artificially as acts of river training to modify the course of the river (fig. 3.6). Loop cuts become succesfully effective if made at fully developed loop, physical indication of which is that the prop and cut alignment will be tangential to the main direction of flow at the two ends of the loop. Under favourable conditions, a lead channel cut along the alignment would develop itself hydraulically.

The short cut (loop-cut) can be made using two different methods. The first method consists of dredging the full ultimately expected cross-section with a bed level corresponding to the average of bed elevations of upstream and downstream transition points. After this closure of the old channel is possible. The other method is to dredge a pilot channel, leaving the remaining clearing to the river itself. The cross-section and the longitudinal profile of the pilot channel should be such that there is sediment transport even at low stages.

The immediate influence of a loop cutoff will be noted in several adjacent meanders due to the local change in slope.
Experience has shown that a stable channel requiring minimum maintenance can be obtained in the stream only by establishing a controlled channel width, proper alignment and optimum bend radii. The primary requisite of alignment is that it be smooth and free from bends that are either too abrupt or too flat or other irregularities in the bank line. The existing meander pattern, the overall slope and channel length should be generally maintained; however, within these limits the channel changes or cutoffs may be made as desired to obtain an optimum alignment.

Straight reaches should be avoided. The straightening of a truly meandering channel over long reaches should never be attempted, even though the anticipated rewards in terms of increased capacity are tempting unless one fully understands and is prepared to accept the consequences.

3.5 Island

An artificially constructed island or an existing natural island (fig.3.7) trimmed to required streamline shape and protected by pitching (revetment) on all sides causes deepening of the river bed in the vicinity through scouring and thereby attracts the river towards itself and holds it permanently. Pitched islands thus form an effective river training tool.

Islands having upstream tips formed into fish-head shapes with necessary revetments are used to maintain definite ratio of discharge of bifurcating channels (ana branches) in braided rivers.

3.6 Retards

Retards (also termed pallasiding) are structures longitudinal to the flow and constructed in the river infront of the bank to be protected (fig. 3.8). Retards have retarding effect on the current that would otherwise hit the bank and cause erosion. Under favourable conditions the space between the retard and the bank gets silted up. Usually fences of wire mesh, barbed wire or other suitable fencing material are formed on the river side face of the retards. The structures can be made submersible too.

Retards may be mabe by,

- Timber pile; single, double or triple rows
- Steel or timber jacks; one or more rows
- Steel, timber or concrete tetrahedrons
- A single or double line of piles and G.I. barbed wires or G.I. wire mesh fencing

3.7 Porcupine

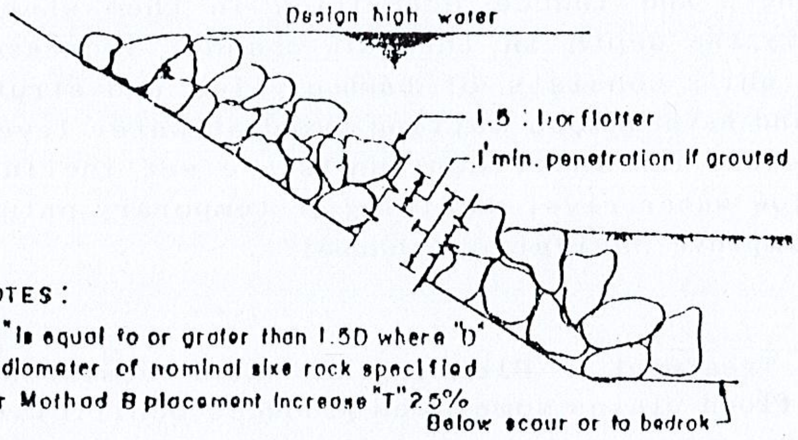
These are box like bamboo frames with leg extensions of framing bamboo units from all corners (fig. 3.9). Porcupines are ballasted by filling the box with brickbats and sunk or placed in predetermined positions. Porcupines are deployed in staggered arrangements in rows and columns to cover the under water surface of slope and toe of river bank desired to be protected from erosion.

Porcupines are effective in sediment laden tidal rivers of coastal area where velocity of flow are moderate. For rivers with deep scour depths, higher flow velocity and turbulance and coarser sediment content, porcupines are not effective. However, porcupines in fruitful treatment usually need replenishment in next seasons.

3.8 Other Methods

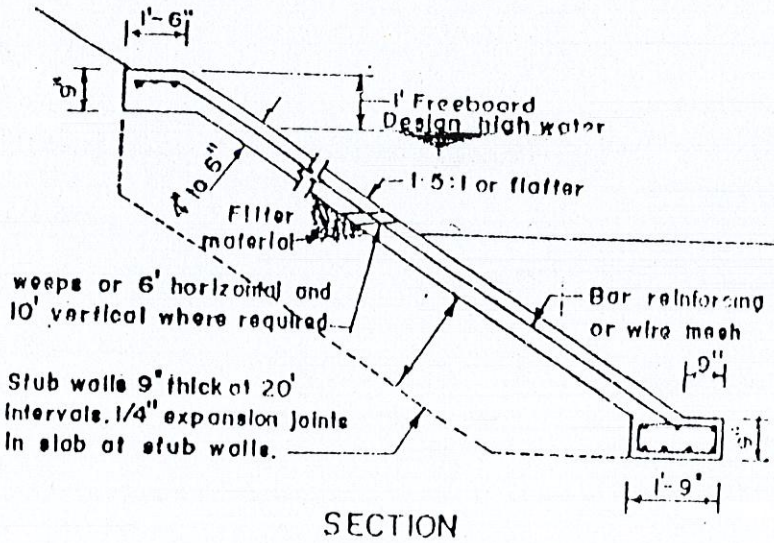
Bandals : Bandals are spur like bamboo fence units constructed in groups accross side channels in river bed to check flow through these channels and induce deposition in them where by flow and consequently the depth in the main channel increases (fig.3.10). The bandal units consists of bamboo piles and struts driven into river bed and have bamboo matting fixed at water levels with bottom remaining open. The individual units are set inclined downstream. This is a low water river training of temporary nature and is used mainly to improve navigation channel.

Biological Treatment : Planting of water resistent trees along reaches of flood plains sometimes produces benificial effect by way of increasing flow resistance along the flood plain (fig.3.11). Combinations of effects of such biologically treated flood plain reaches help training the river flow in desired cases.



NOTES :
"T" is equal to or grater than 1.5D where "D" is diameter of nominal size rock specified
For Method B placement increase "T" 25%

SECTION
FIG. 3·1

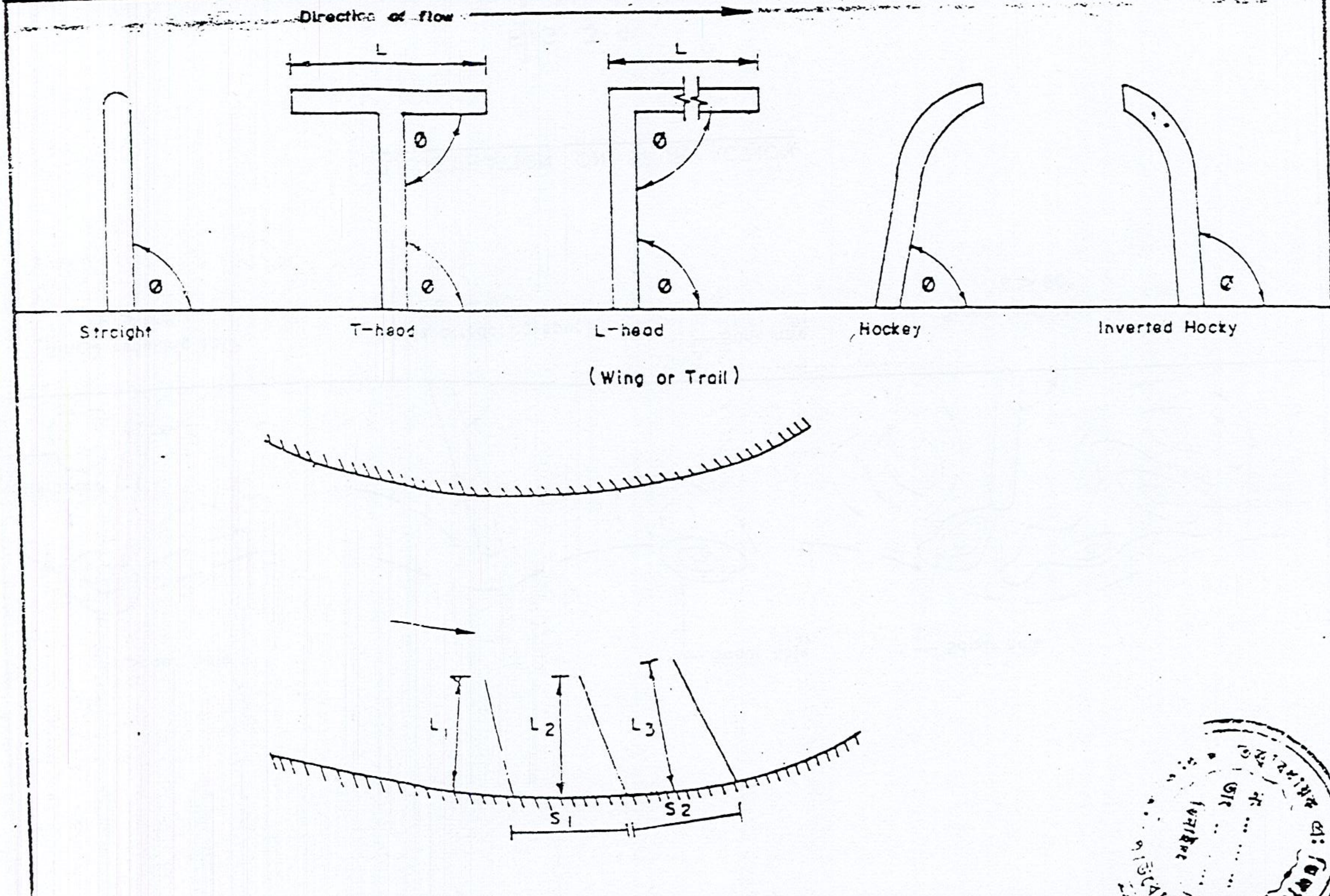


SECTION
FIG. 3·2

NOTES :-
Detail shown for stable soil conditions. For unstable conditions modify footing detail.
Rough or smooth surface finish to be specified according to hydraulic requirments.
Concrete paving may be applied as air-blown mortar.

Direction of flow

Direction of flow



DEFINITION SKETCH FOR SPURS

FIG. 3·3

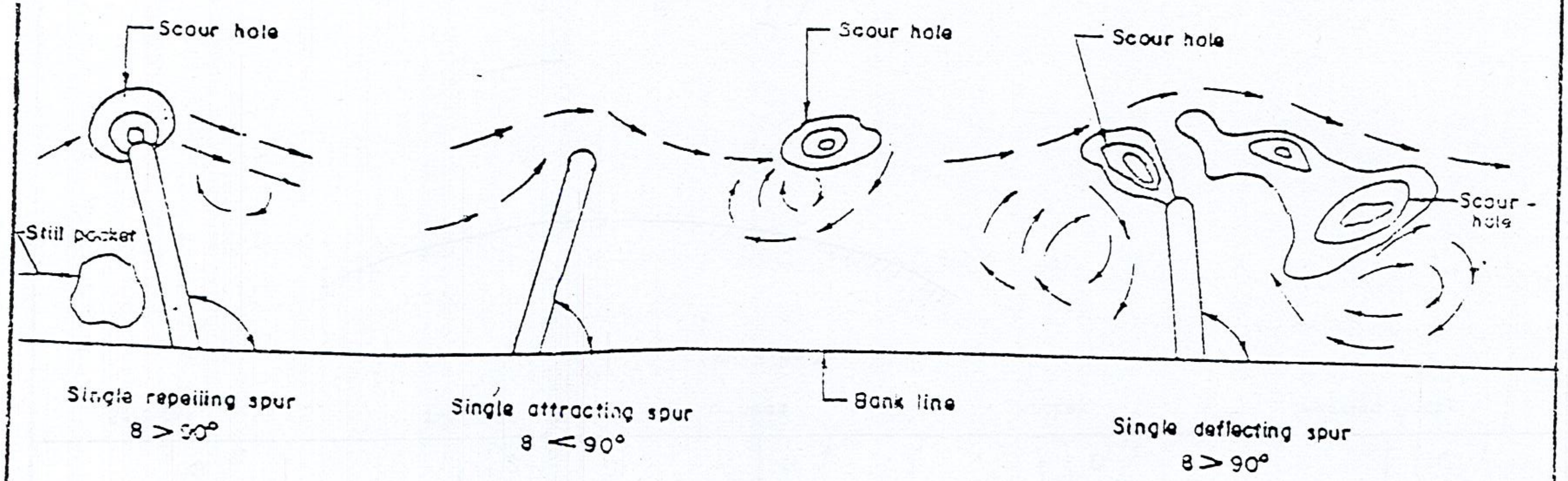
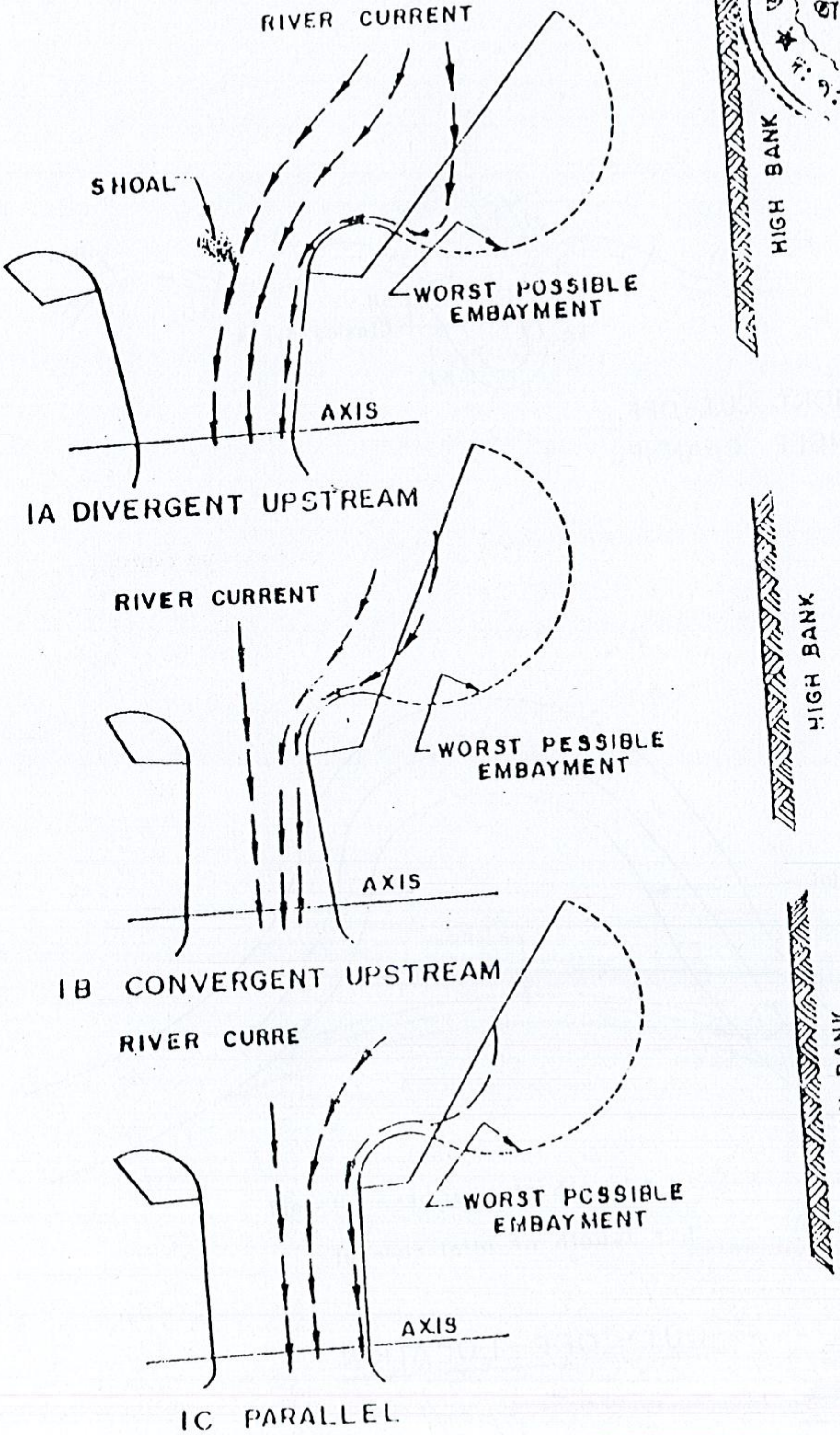
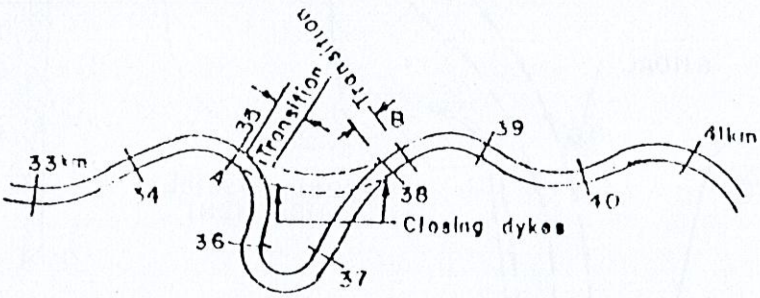


ILLUSTRATION ON SPUR ACTION

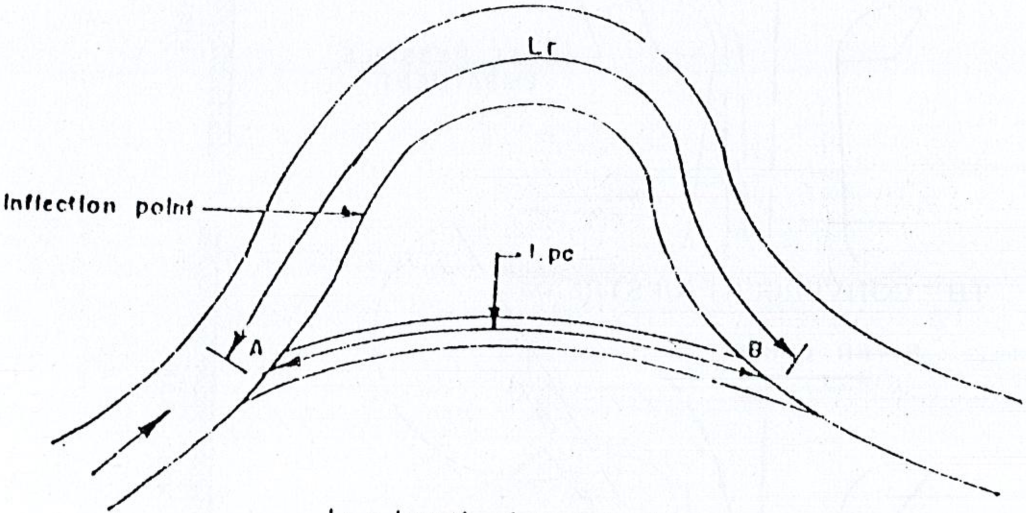
FIG. 3·4



DIFFERENT FORMS OF GUIDE BANKS
FIG. 3·5



SHORT CUT-OFF
SINGLE CHANNEL

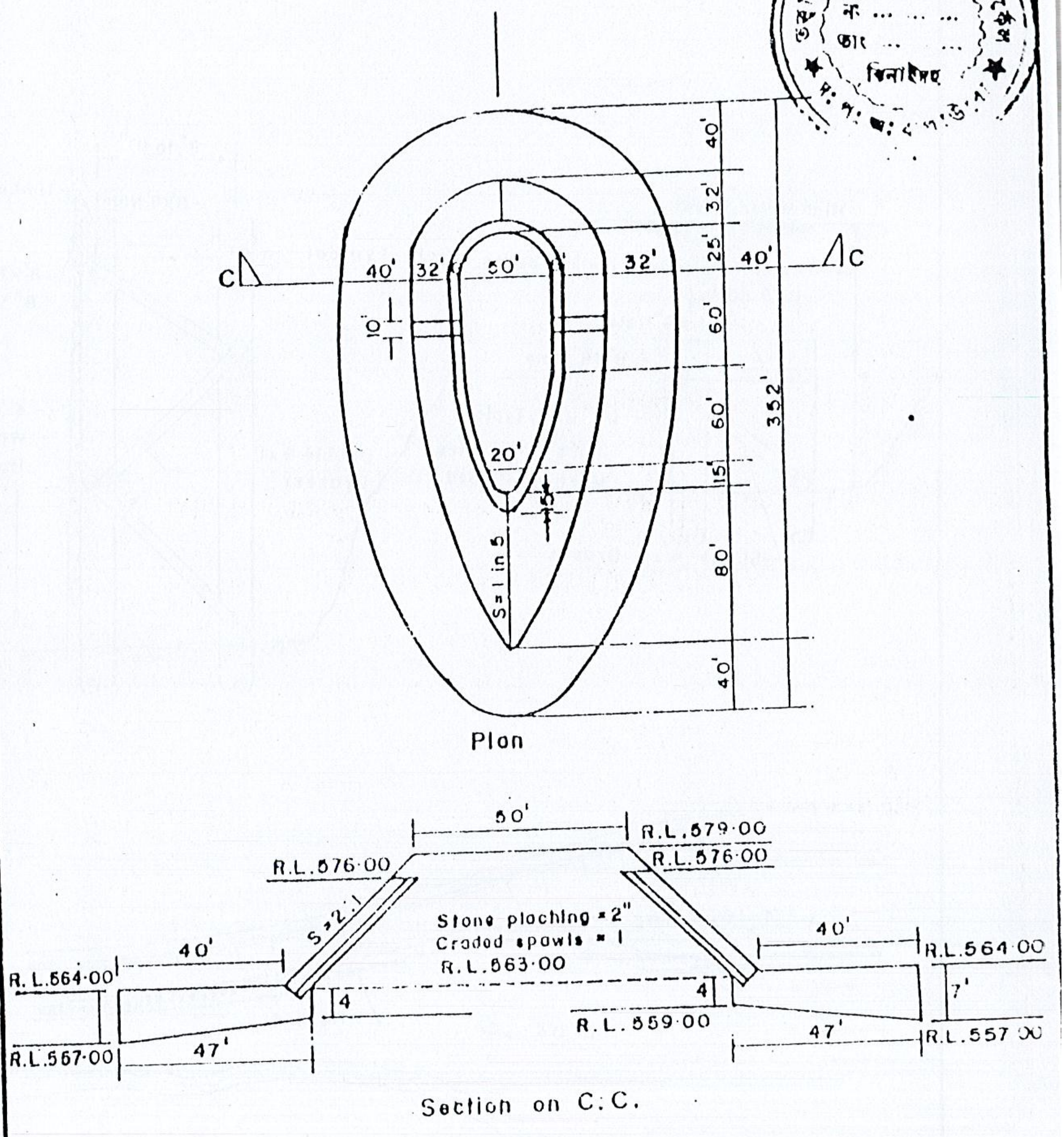


Lr = length of river bend cut-off

L = length of pilot channel

CUT-OFF LOCATION

FIG-3·6



PLAN AND SECTION OF PITCHED ISLAND CONSTRUCTED AT R.D. 4500 UPSTREAM SULEIMANKI WEIR, SUTLEJ RIVER.

FIG. 3·7

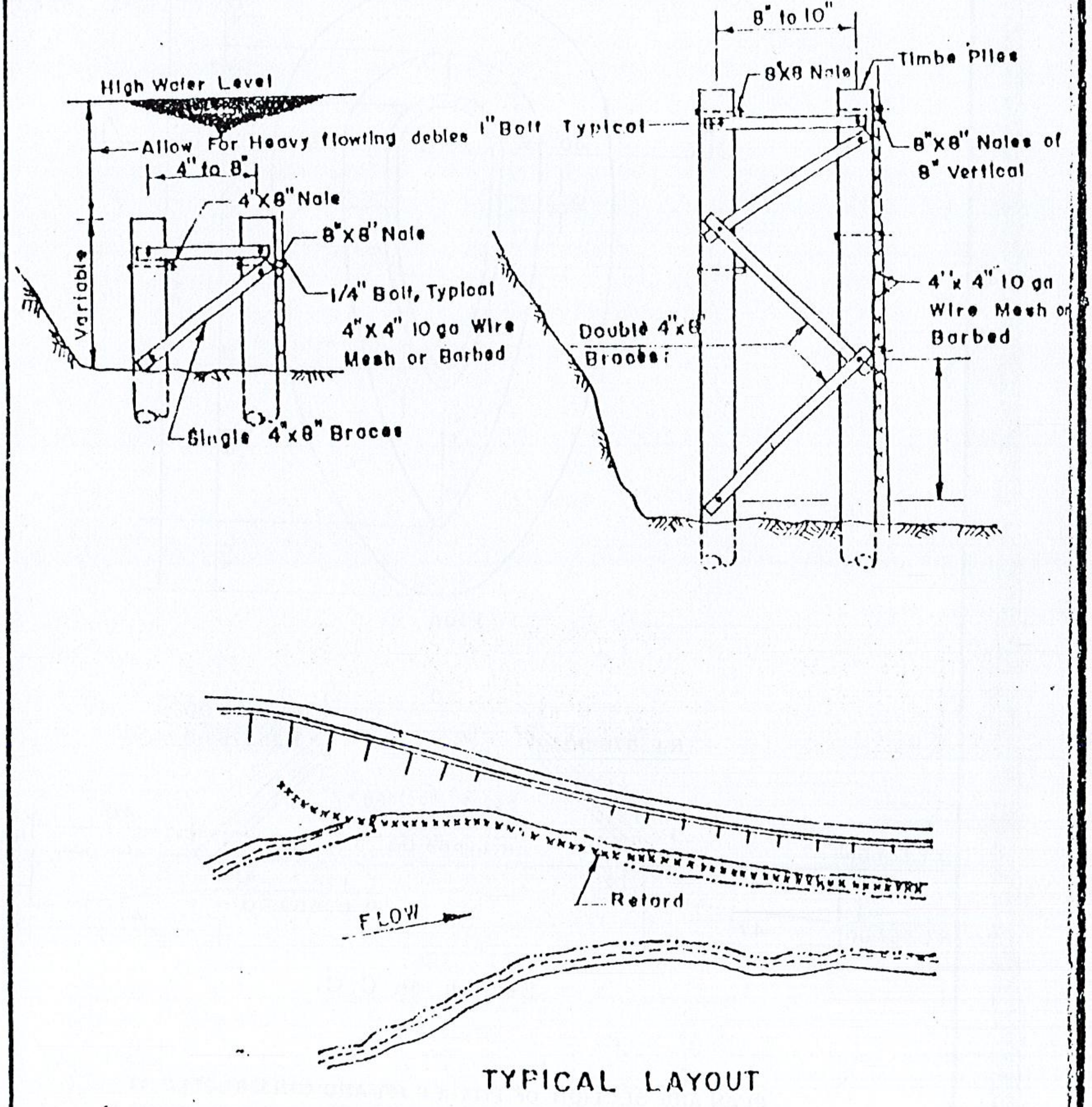
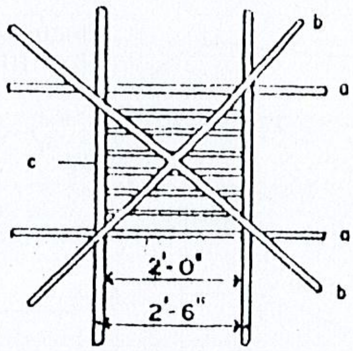
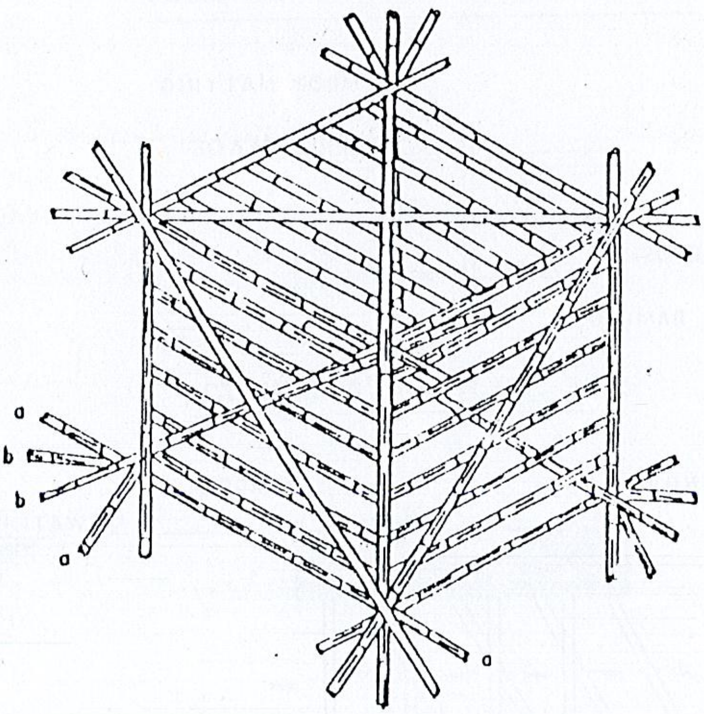


Fig. 3-8 Bank and shore protection by pile retard

FIG. 3-8



PLAN AND END ELEVATION
OF PORCUPINE



ISOMETRIC VIEW OF PORCUPINE

Fig. 3·9 Porcupine

(Source : BWDB Typical Drawing)

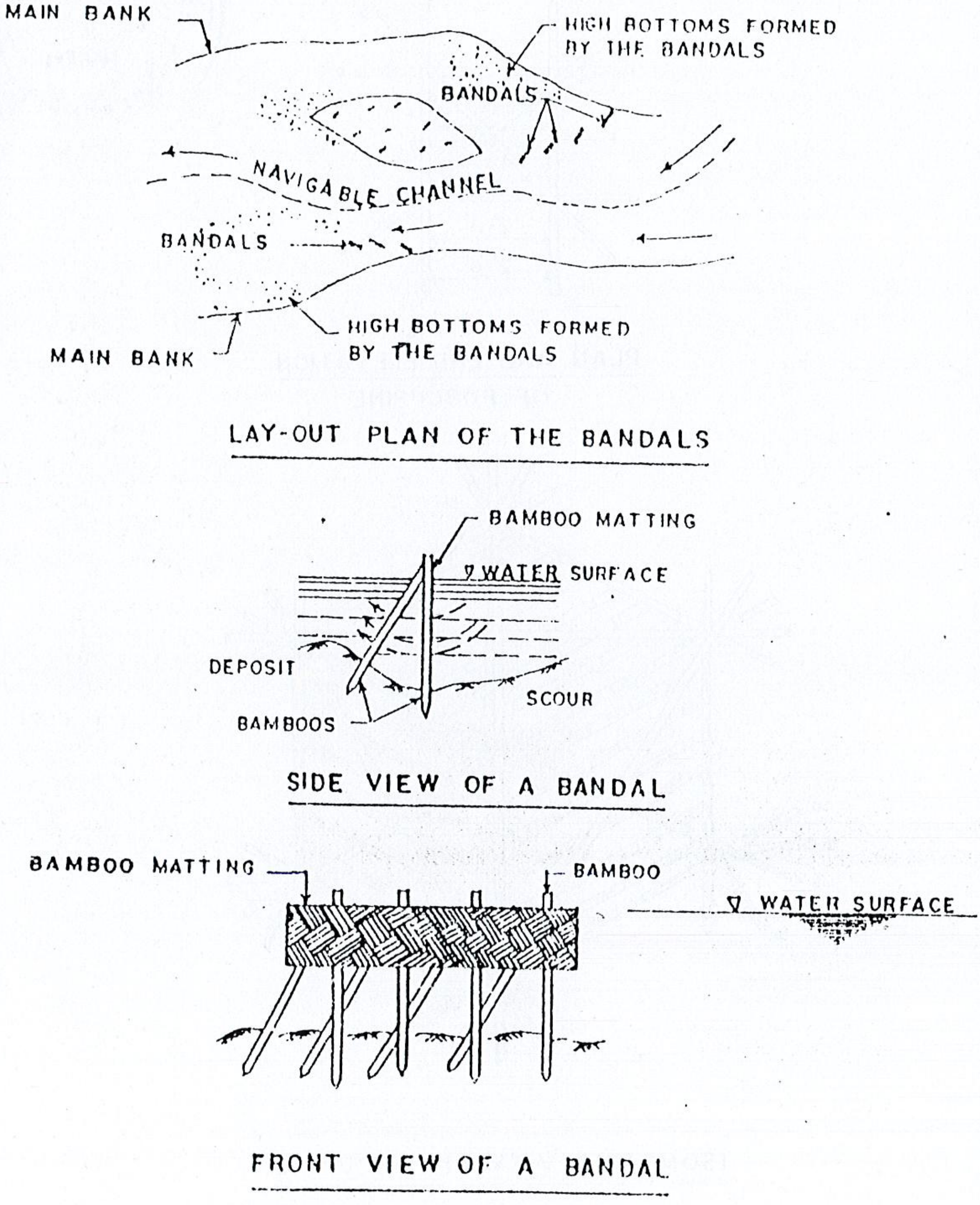


Fig. 3·10 Plan and Elevation of bandal
(Source: Nishat, A., 1986)

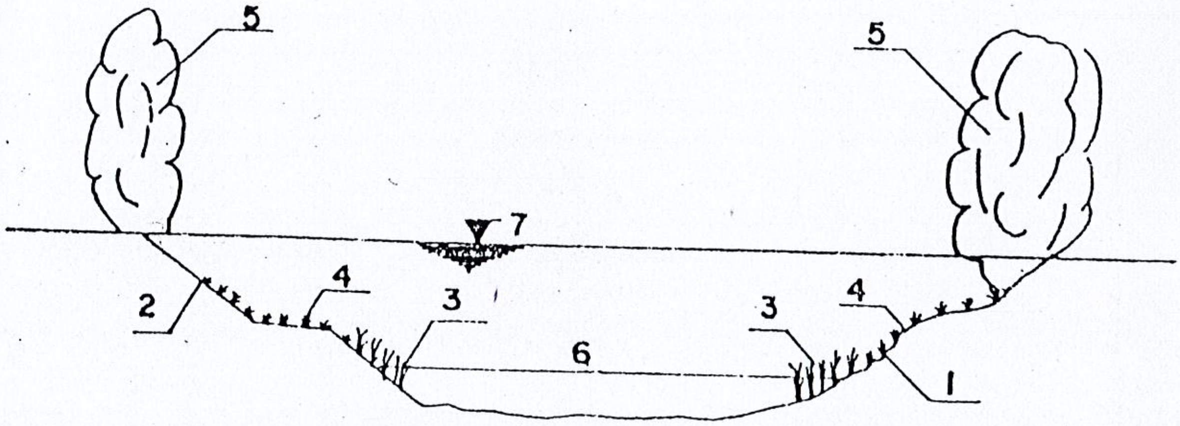


Fig. 3·11 Vegetation zones on a river bank 1. old bank, 2. new composite 3. reed, 4. grass, 5. Trees and shrubs, 6. mean water level 7. high water level.

GUIDE TO PLANNING AND DESIGN OF
RIVER TRAINING AND BANK PROTECTION WORKS

CHAPTER 4

HYDRAULIC FACTORS

CHAPTER 4

4.0 HYDRAULIC FACTORS

For design of structural element of river training works, hydraulic factors such as water level, discharge, flow velocity, wave height and anticipated scour bed level of river have to be established. The criteria and considerations for selecting or determining each of these hydraulic factors are described in the following sections:

4.1 Water Level

The design water levels at river training works are required for the following purposes :

- to decide the boundary between the above water slope protection and below water slope protection.
- to fix the highest level of protection works.
- to compute scour depth.

The design water levels may be taken from available analysed records or may be computed from available data. If the gauge station is located away from the proposed training works, the water level data need to be established by simple gauge correlation, or to be predicted by use of complex Mathematical Model. The following end result of hydrological analysis are important for design purpose ;

Flood levels - 1:100 Years
1:50 Years
1:20 Years
Bankfull level
Average year level

Low water Level - Lowest during dry season
- Average lowest in various dry months

4.2 Discharge

For design of river training works, the discharge is required for following purposes :

- to compute scour depth which is in turn is used to select design scoured bed level.
- to determine average velocity of flow which is in turn is used to select local velocity at protection site.

The design discharge may be taken from available analysed records or may be computed from available data. In case of ungauged basin the same may be computed by simple synthetic unit graph method or by complex Mathematical Model. The design data which are important are as follows :

Flood Discharge	:	1 in 100 Years
		1 in 50 Years
		1 in 20 Years
		Average year Discharge
		Bankfull Discharge

The design discharge for various conditions may be taken as follows:

- For major rivers : 1 in 100 year or 50 year discharge.
- For minor rivers : Bankfull Dischare
- For flashy rivers : Dominant discharge which may be bankfull discharge or 70% of 1 in 20 year discharge whichever is higher.

4.3 Current

The revetment used to protect an erodable surface against erosion has to withstand the drag forces generated by the velocity. Although the local velocity is related to the velocity in the channel at some distance from the revetment, the most influential factor on the local velocity is the boundary geometry in the immediate neighbourhood.

The velocity close to rough and smooth boundaries can be predicted from semi-theoretical equations, but the same capability does not exist for the case of irregular boundarries. The usual practice in design is to apply rules of thumb to estimate local velocities. Thus, the local velocity at the boundary of a straight channel is assumed to be two thirds of the cross sectional mean velocity The velocity at the outer bank of a severe bend can be upto four thirds of the cross sectional mean.

No data are available for the velocities around the noses of groynes or cross bars. Neill suggested much higher factors than that for a river bend need to be applied in order to estimate the velocities at such locations. It has been shown both theoretically and experimentally that the local velocity around the periphery of a cylindrical pier in a channel is twice the mean velocity in the channel. In the absence of any other field or experimental data, it is reasonable to assume that the revetment at the nose of groynes should be of a size that will be stable against velocities that are at least twice the mean velocity upstream.

The data of average velocity may be obtained from measured data of a discharge measurement site. Otherwise, if the discharge and corresponding water levels are known, the average velocity may be computed from surveyed river cross section at point of interest.

4.4 Waves

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

- Runup of waves against slope which might overtop the upper limit of protection
- Erosive forces of breaking waves against the slope causing erosion

The wind climate is widely recognised as dominated by the "Nor wester" squalls. These squalls are quite small disturbances, associated with thunderstorms. They are most common in March-April, but can occur late in the year.

The classic pattern is a rapid rise in wind velocity accompanied by a rapid swing of wind direction, followed by a progressive decay of wind speed. There is, however, quite a range of variation; the higher speeds are probably associated with shorter durations and there may be a continuous range into tornado squalls which would only last a few seconds.

The peak gust speeds v (m/s) that may occur during the passage of a "Nor-wester" could be related to a return period T (year) by the following relation :

$$V\ (\text{thunderstrom}) = 30 + 4.5 \ln (T) \qquad (m/s)$$

Apart from "Nor- westers" there is the risk of encounter of a tropical cyclone. The classic pattern would be a cyclonic depression with high speed over a diameter of some 80 km, moving at some 4 m/s (and thus taking a few hours to pass). A "severe cyclone" as defined in the Bangladesh Atlas, will only come within range (i.e. 40 km from the centre) about once in 30 years. The best present estimate is that the associated gust speed would be 33 m/s.

The principal factor affecting the design of slope protection is wave action. The mechanics of wave generation are extremely complex, and the forces causing erosion during wave attack on an earth slope are both varied and complex. The described ranges of riprap design assume that the wave height is a direct measure of the erosiveness of the wave attack (fig.4.1).

To evaluate wave height the following factors that create waves in open water must be analyzed:

- Design wind direction
- Effective fetch
- Wind velocity and duration

For many years wave height have been estimated by the empirical formulas of Stephenson, Molitor, Creager and others. In recent years more exact methods have been developed by theoretical and experimental analysis. Figure 4.2 present the generalised correlations of significant wave height (H_s) and wind duration (T_d) with design wind velocity (U_d) and effective fetch (F_e).

The relationships are based on the Sverdrup-Muck theory as modified by Bretschneider and are considered accurate and reliable enough for riprap design.

Figure 4.2 is based on so-called "deep water waves", which are defined as waves having lengths equal to or less than twice the depth of water. In most instances deep water waves are generated in front of an embankment.

Figure 4.2 can be used to estimate the significant wave height (H_s), after determining the design wind direction, effective fetch, design wind velocity over water and minimum wind duration.

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

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

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

- Wind Velocity and Duration (U_d & T_d): Reliable estimate of the maximum wind velocity that would exist over a length of time at a given site is practically impossible. However, value of wind velocity can be obtained from climatological data of the area for cyclone, thunderstorms or norwester and design wind velocity may be selected for appropriate return period on importance of protection.

- Significant Wave Height (H_s): Significant wave height is the average height of the highest one-third of the waves for a specified period of time. The significant wave height (H_s) can be determined from figure 4.2, by use of computed/obserbed design wind velocity (U_d) and the effective fetch (F_e).

The following table summarises the characteristics of the waves that will be generated for variety of conditions:

Table 4.1

Characteristics of the Waves

Wind speed (m/sec)	Minm duration of wind (hrs)	Fetch length (km)	Wave height (m)	Wave period (secs)
16	1.00	5.0	0.7	2.8
	1.75	10.0	0.9	3.3
	2.25	15.0	1.2	3.8
30	0.75	5.0	1.3	3.5
	1.50	10.0	1.8	4.5
	2.00	15.0	2.0	5.0

Wave heights are measured from trough to crest, and the values in the table apply to deep water waves i.e where $h/L_0 > 0.25$

h = water depth

L_0 = wave height in deep water

$= gT^2/2\pi$

T = wave period

In shallow water ($h/L_0 < 0.25$), the wave height will increase.

4.5 Scour

The protection of toe of any river training is of prime importance as regards to the stability of overall slope protections against scour. The main purpose of analysis for scour is to estimate the maximum scour depth for the proposed river training sites which can then be used to generate design conditions.

Scour is a lowering of channel bed below an average assumed level generally the result of secondary currents or vortices that occur in conjunction with river features such as bends, at abrupt changes in flow direction, obstructions, constrictions, confluences, control structures and piers of various types and sizes. The severity (depth) and extent of scour is dependent upon the strength of the secondary currents developed at the concerned river features.

'Depth of Scour' refers to the material removed below the stated channel bed. 'Scoured depth' refers the depth of water above a scoured bed under stated flow conditions.

A distinction can be made between general scour and scour that occurs more locally. General scour is the reaction of the river on changes its boundary conditions, like aggradation or degradation owing to accelerated soil erosion, sea level rise, cuttoffs of bends etc. More localized scour can be ditinguished in a number of different types, notably :

(1) constriction scour, caused by a local constriction of the width of a river;

(2) bend scour, occuring along the rivers, and being characterized by deep scour holes together with a point bar in the inner bend;

(3) confluence scour, occuring in the reach downstream of the junction of two river reaches, like in the case of the branches of braiding river or of the confluence of a tributary with a main river;

(4) protrusion scour, occuring when the bank of the river protudes into the channel, like in the case of a local strong point (a non erodable reach);

(5) local scour, occuring near bank protection works like groynes and revetments.

In figure-4.3, the different types of scour are schematically indicated.

In principle there are two different ways for estimating the scour depth near structures. The first method is based on the regime approach. For the river under consideration the regime depth is estimated with an appropriate formula. Next this regime depth is multiplied with a cofficient, that essentially is based on experience. Hence, the different types of scour listed above are not explicitly accounted for (any computed), but the local conditions are reflected in the estimated values of the multiplication coefficient which indeed depend on local conditions.

This first method has been used extensively in the Indian Sub-continent. The second method is a more recent development, based on the increased understanding of river processes and local scour phenomena. This method attempts to distinguish between the different types of scour, and for each type a quantative estimate is made. In a second step the different values are combined to arrive at the combined scour depth that will actually occur.

Lacey's regime formula is widely used in this sub-continent to find out scour depth in unconstricted alluvial rivers. this emperical regime formula due to Lacey is given below :

$$D_s = 0.47 \, (Q/f)^{1/3}$$

where,

D_s = the scour depth at design discharge

Q = the discharge

f = Lacey's silt factor

= $1.76 \, (d_s)^{1/2}$

d_s = weighted mean diameter of sediment particles in mm.

The water surface level corresponding to Q must be known in order to determine scour levels from the above equation. The above equation gives only an estimated mean depth accross the channel section. To estimate the maximum natural scoured depth, a multiplying factor must be added. The table shown below gives co-efficients recommended by the Indian Code (after Lacey) based mainly upon consideration of cross sectional shape.

Table- 4.2

Emperical Multiplying Factors for Maximum Scoured Depth

Nature of Location	Factor
Straight reach of channel	1.25
Moderate bend	1.50
Severe bend	1.75
Right angle abrupt turn	2.00
Noses of piers	2.00
Alongside cliffs & walls	2.25
Noses of guide banks	2.75

In working out the silt factor f, it is important to see as a which sand sample it pertains. Supposing the surface sand from the river bed has the f value 1.00 giving a depth of scour 100 ft at which depth the sand met with has a silt factor f=0.80, i.e. finer than the surface sand, then, evidently, the depth of the scour will be more than 100 ft. For arriving at a more accurate scour depth, the silt factor used should be such that it conforms with the nature of the sand of the anticipated scour depth.

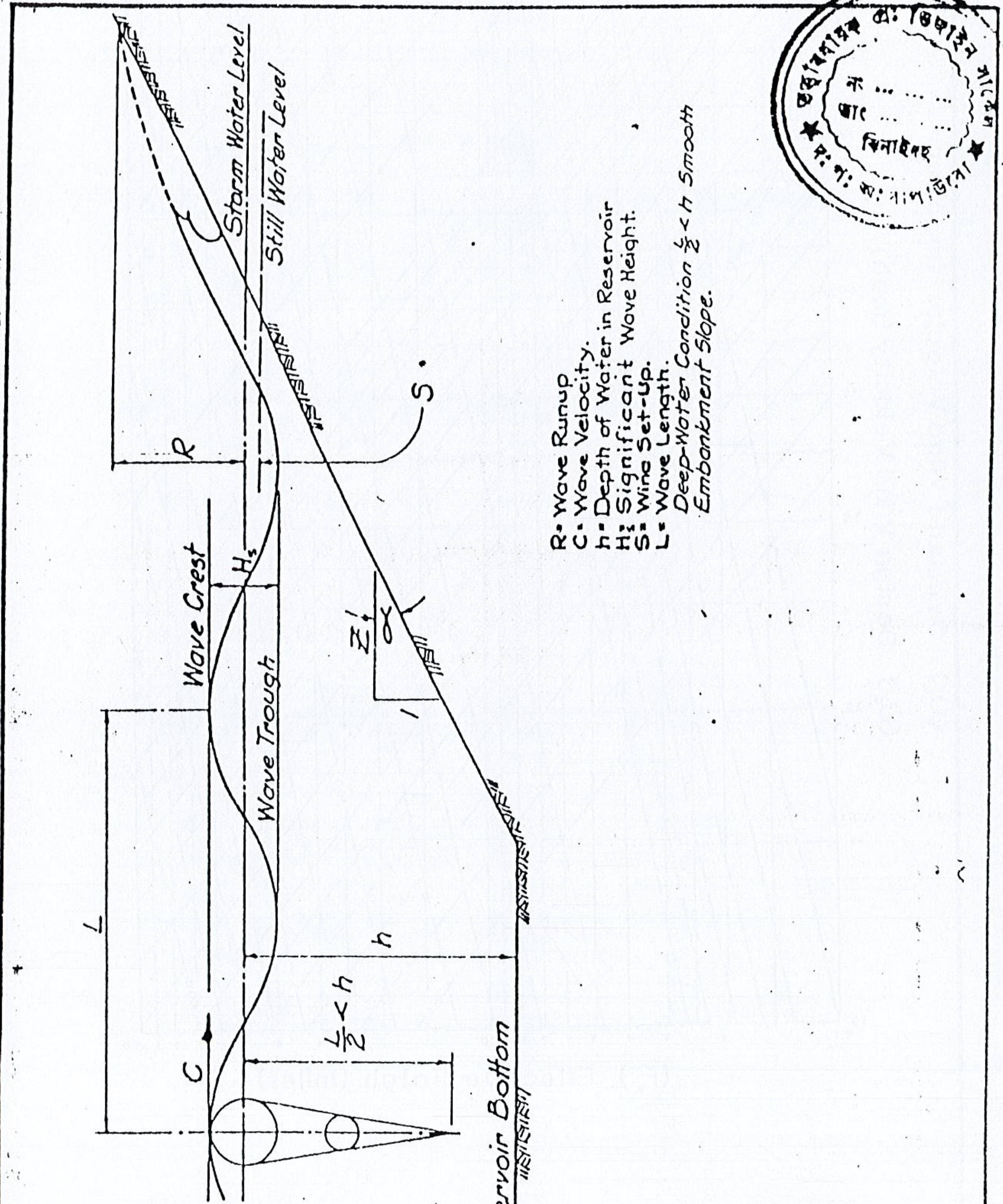
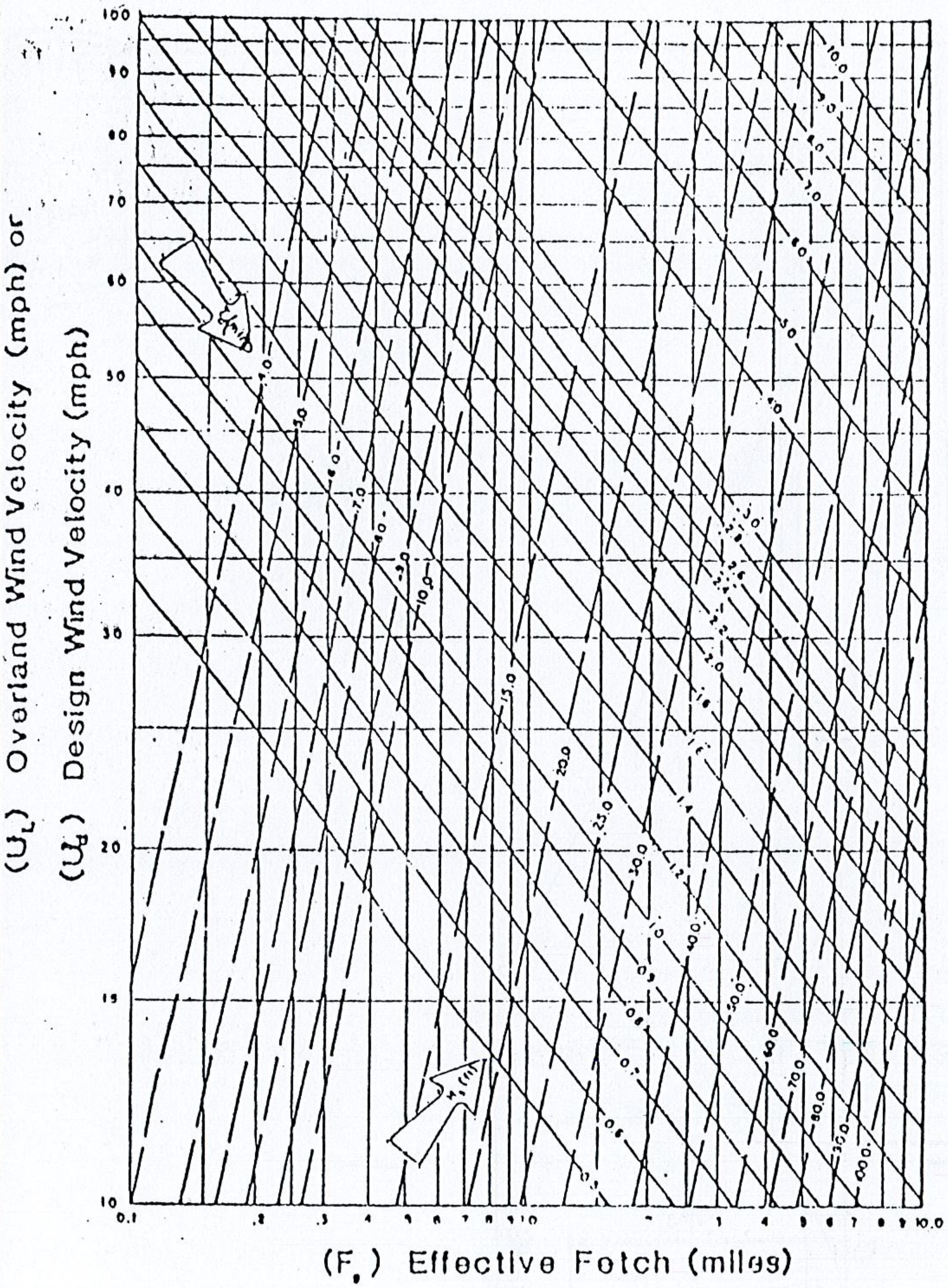


FIGURE - 4·1

ILLUSTRATION OF DEFINITIONS

WNTC ENG. STAFF



REF. (2)

Equation; (A17) & (A18) From Appendix A

$$\frac{gH_s}{U^2} = 0.0026 \left(\frac{gF_e}{U^2}\right)^{0.47}$$

$$\frac{gT}{U} = 0.46 \left(\frac{gF_e}{U^2}\right)^{0.28}$$

FIGURE. 4·2

GENERALIZED CORRELATIONS OF SIGNIFICANT WAVE HEIGHTS WITH RELATED FACTORS

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PORTLAND, OREGON

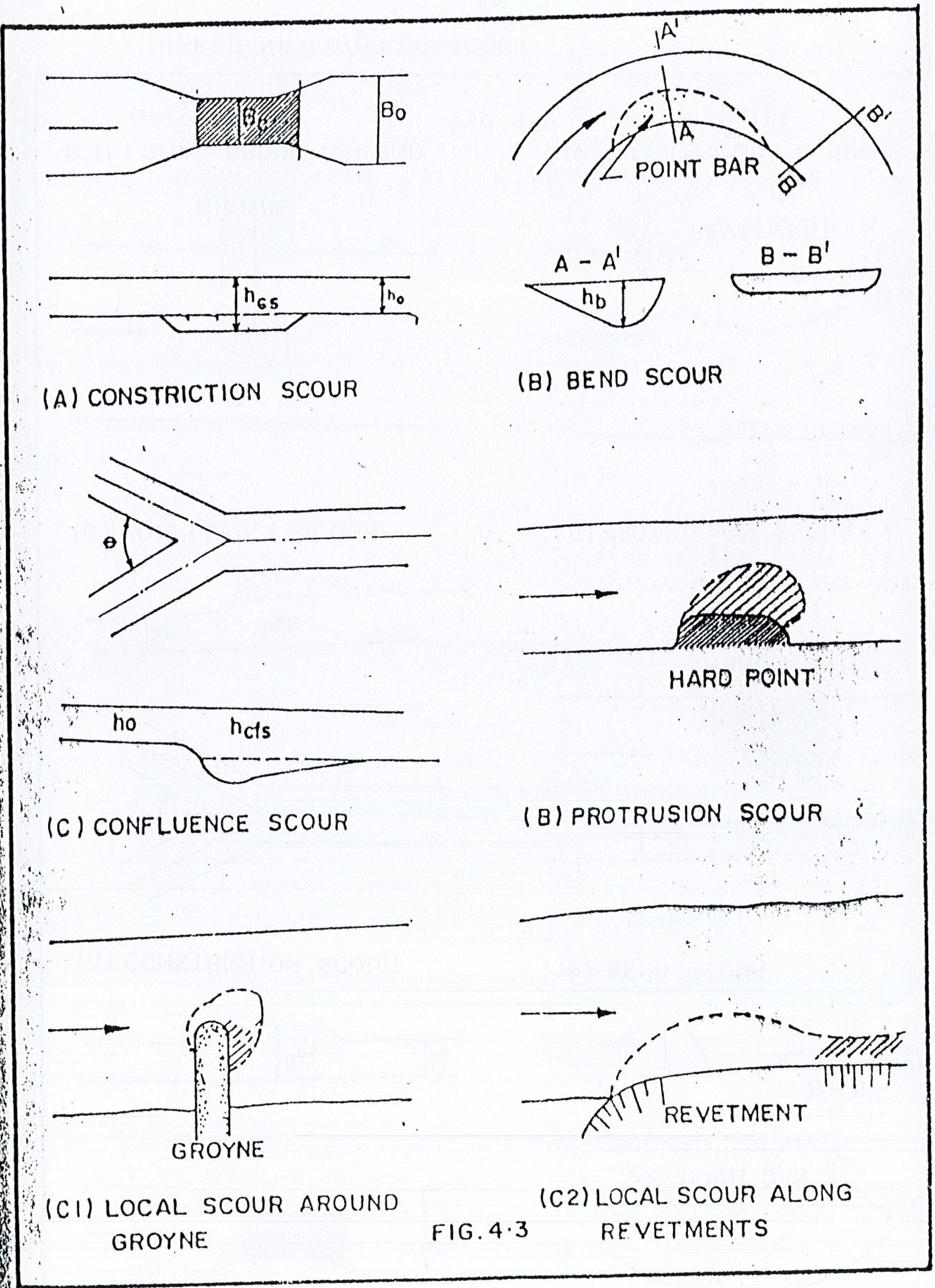


FIG. 4·3

DIFFERENT TYPES OF SCOUR

CHAPTER 5

5.0 GEOLOGICAL, SEISMIC & GEOTECHNICAL ASPECTS

5.1 Geological Factors

Bangladesh consist primarily of deltaic alluvial sediments of three great rivers the Ganges, the Brahmaputra and the Meghnan and their tributaries. According to Morgan and McIntre, the entire Bangladesh is a part of the Bengal basin filled in the Tertiary-Quaternary geological period. The thickness of sediment cover over the basement rock, starting from about 180m along the Rangpur-Dinajpur axis, increases south eastward to over 12000m in the eastern part of the country.

Geologically the land can be divided into three broad physiographic regions. These are the Tertiary Hills, the Pleistocene Uplands and the Recent Plains (fig. 5.1). This tripartite division also coincides with the major relief features of the country. The Recent Plain can be further subdivided into Piedmont plain, Flood Plains and the Tidal and Estuarine Flood Plains (fig. 5.1).

5.2 Seismic Factors

Over 200 major earthquakes occurred in and around Bangladesh between August 1833 and July 1971, but there seems to be no seismically active fault in the territory. However, the causative faults and regions of high seismic activity exist to the north and east of Bangladesh in neighbouring India and Burma, and earthquakes in these areas affect the adjacent regions in Bangladesh as well. Recognising this the Committee of Experts on Earthquake Hazard Minimization has published a report entitled "Seismic Zoning map of Bangladesh and Outline of a Code for a Earthquake Resistant Design of Structures". The Committee recommended Bangladesh to be sub-divided into three zones; Zone-I, Zone-II and Zone-III (fig. 5.2). The report suggests the basic horizontal seismic co-efficient of 0.075, 0.15 and 0.25 for zone-I, II and III respectively.

5.3 Geotechnical Factors

Failure in a soil mass results from exceeding its ultimate shear resistance. Basic components comprising the shear resistance are cohesion and frictional resistance generated by the interparticle forces. The latter are a function of effective stress.
Saturation in conjugation with shear movement may dramatically affect the magnitude of developed effective stress and hence shear resistance.

Without any other effect of the presence of ground water other than buoyancy it is commonly accepted that the angle of internal friction ϕ will approximately define an infinite slope angle with a safety factor $n = 1$. However, external loading e.g. due to earthquake loading and seepage forces, may render such a slope unstable. Therefore, Standard Code of Practice recommends stable slope to be designed for a minimum safety factor $n = 1.5$.

Because many actual slopes are barely stable with factor of safety little above 1.0, any unusual actions that increase destabilizing forces or reduce strength- such as strong earthquakes, unusually heavy rainfall, broken water lines, loading the upper parts of slopes or undercutting lower parts- can trigger slipouts or landslides. Improving the stability of earth slopes with drainage is one of the most important activities of civil engineers.

5.3.1 Phenomena Governimg Slope Stability

The normal regime of a river with falling and rising water levels affects ground water table in the adjoing land.

Without any ground water flow between ground and free water the hydraulic gradient equals zero. Overburden stresses will only govern shear resistance. An in or outwardly directed gradient perpendicular to the slope face will increase or reduce the overburden stress level. For above loading conditions and consistency of fine and silty sand layers, a potential failure plane will have an approximately circular shape.

Stable earth slopes, both natural and man-made, are of great importance to mankind. Although many landslides occur in slopes because of natural influences, human activities such as undercutting, piling earth on unstable slopes, or raising the ground water level by constructing reservoirs, are important causes of landslides. Unfavourable ground water and seepage conditions are among the most frequent. Water lowers stability and contributes to slope failures in the following ways:

- By reducing or eliminating cohesive strength.
- By producing pore water pressures which reduce effective stresses, thereby lowering shear strength.
- By producing horizontally inclined seepage forces which increase the overturning moments and possibility of failure.
- By lubricating failure planes after small initial movements occur.
- By supplying an excess of fluid that becomes trapped in soil pores during earthquakes or other severe shocks, leading to liquefaction failures.

The following conditional events can be recognized for analysing overall stability :

- With water levels in the river rising, the rise of the ground water table will lag behind. As a result this slope will be subjected to an inwardly directed seepage force. This force, acting perpendicular to the slope, will increase overall slope stability. In conjunction with the erosive action of the river at an outer river bend, a relatively steep and critical underwater slope can then be developed and maintained;

. The body of a developed natural slope may become overste when ground water flow due to subsequent falling water leve in the river (fig. 5.3) results in a reduction of effecti stresses due to uplift forces. Simultaneous reduction of shear resistance will then reduce its safety against slidi and may ultimately, render a slope oversteep;

. Critical slope angles must, by definition, represent a safety factor of $n = 1$

. The presence of a slope protection is required to suppre erosion phenomena and to maintain a required overall slo angle with $n>1$. The protection, however, does neither redu nor increase the macro-stability.

Previously described phenomena lead to the conclusion that next the common erosive action of an exposed slope falling water leve in the river will introduce the most severe risks of loss of micr and macro-stability.

5.3.2 Safety Factor and Procedures for Slope Stability Analysi

The numerical analysis of slope stability should be accompanied the choice of an appropriate safety factor "n", defining the rat of developed over ultimate shear stress. A certain safety fact must be applied for the overall slope (macro) as well as a structural element of the slope protection (micro). The magnitu of "n" is, among other thing, a function of :

. variation in the design parameters, shear strength & densit

. type & frequency of various loading types (self weight, ground water, surcharge on bank and earthquake loading);

. slope deformation and maintenance criteria, the former referring to a predominantly elastic behaviour under permane loading and the latter too a loading condition where some plastic deformation can be accepted.

The Swedish Slip Circle Method, which supposes the surface of rupture to be a cylindrical one, is a comparatively simple method of analysing slope stability. In this method, the factor of safety against sliding is the ratio of the average shear strength to the average shear stress determined by statics on a potential sliding surface. The factor of safety against shear failure along the arc of a circle determined by the slip circle method is as follows :

$$F.S. = \frac{cL + \tan\phi - (N_s - U_s)}{T_s}$$

where,

F.S. = factor safety

T_s = sum of tangential forces tending to produce movement of the soil along the slip surface circumference

N_s = sum of normal forces for all the slices

U_s = sum of uplift pressures

ϕ = angle of internal friction of the soil

L = length of arc intersecting the bank slope

c = cohesion of the soil.

[illegible]

$$F.S. = \frac{cL + \tan\phi \cdot (N - U)}{T}$$

Where,

$F.S.$ = Factor of safety

T = sum of tangential forces tending to produce movement of the soil about the sliding circumference

N = sum of normal forces for all the slices

U = sum of uplift pressures

ϕ = angle of internal friction of the soil

L = length of arc along the sliding circle

c = cohesion of the soil

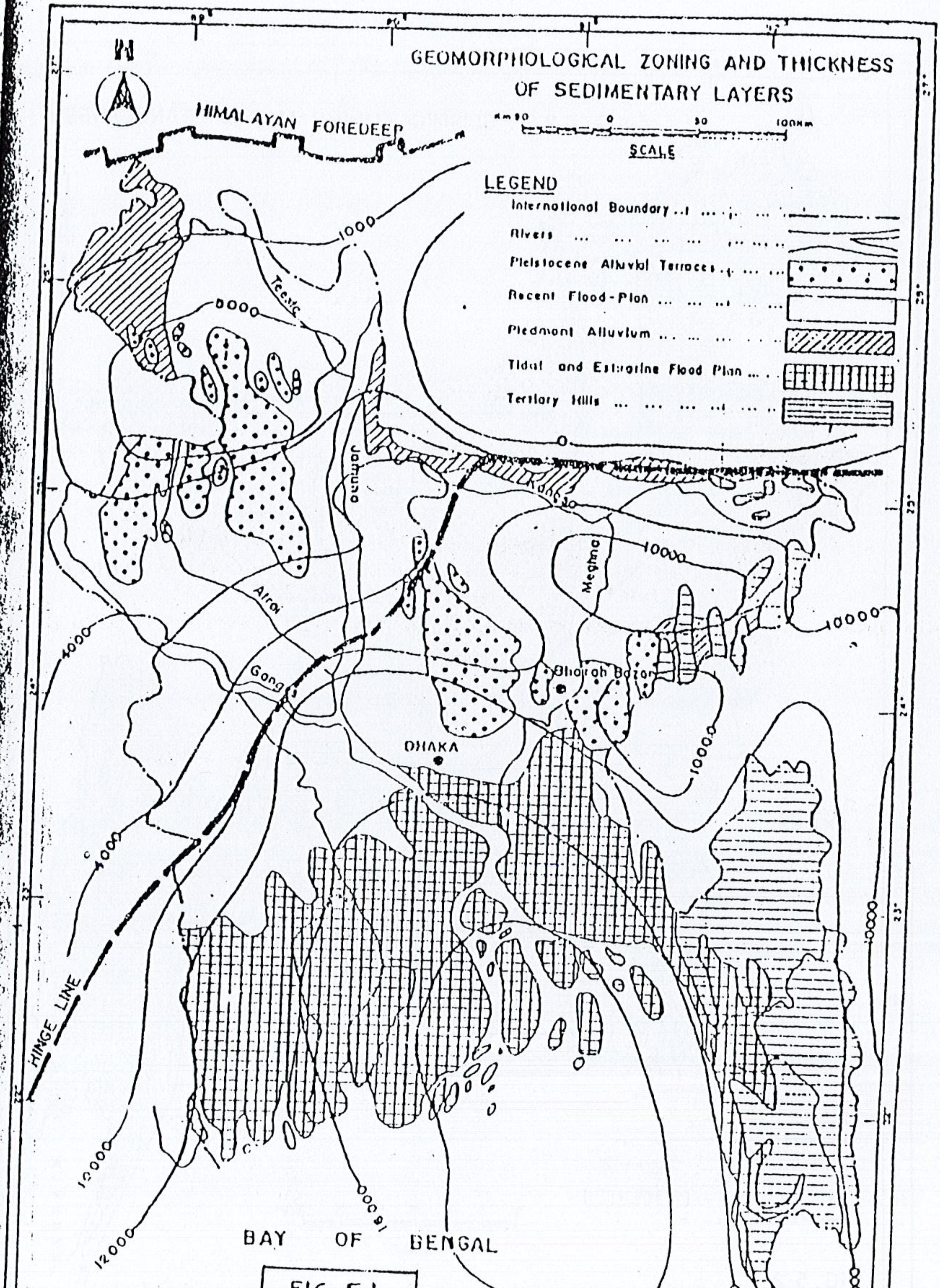

GEOMORPHOLOGICAL ZONING AND THICKNESS
OF SEDIMENTARY LAYERS
HIMALAYAN FOREDEEP
SCALE
100Km
LEGEND
International Boundary
Rivers
Pleistocene Alluvial Terraces
Recent Flood-Plain
Piedmont Alluvium
Tidal and Estuarine Flood Plain
Tertiary Hills
Teesta
Jamuna
Atrai
Ganga
Meghna
Bhairab Bazar
DHAKA
HINGE LINE
BAY OF BENGAL
1000
5000
4000
10000
12000
16000

FIG. 5.1

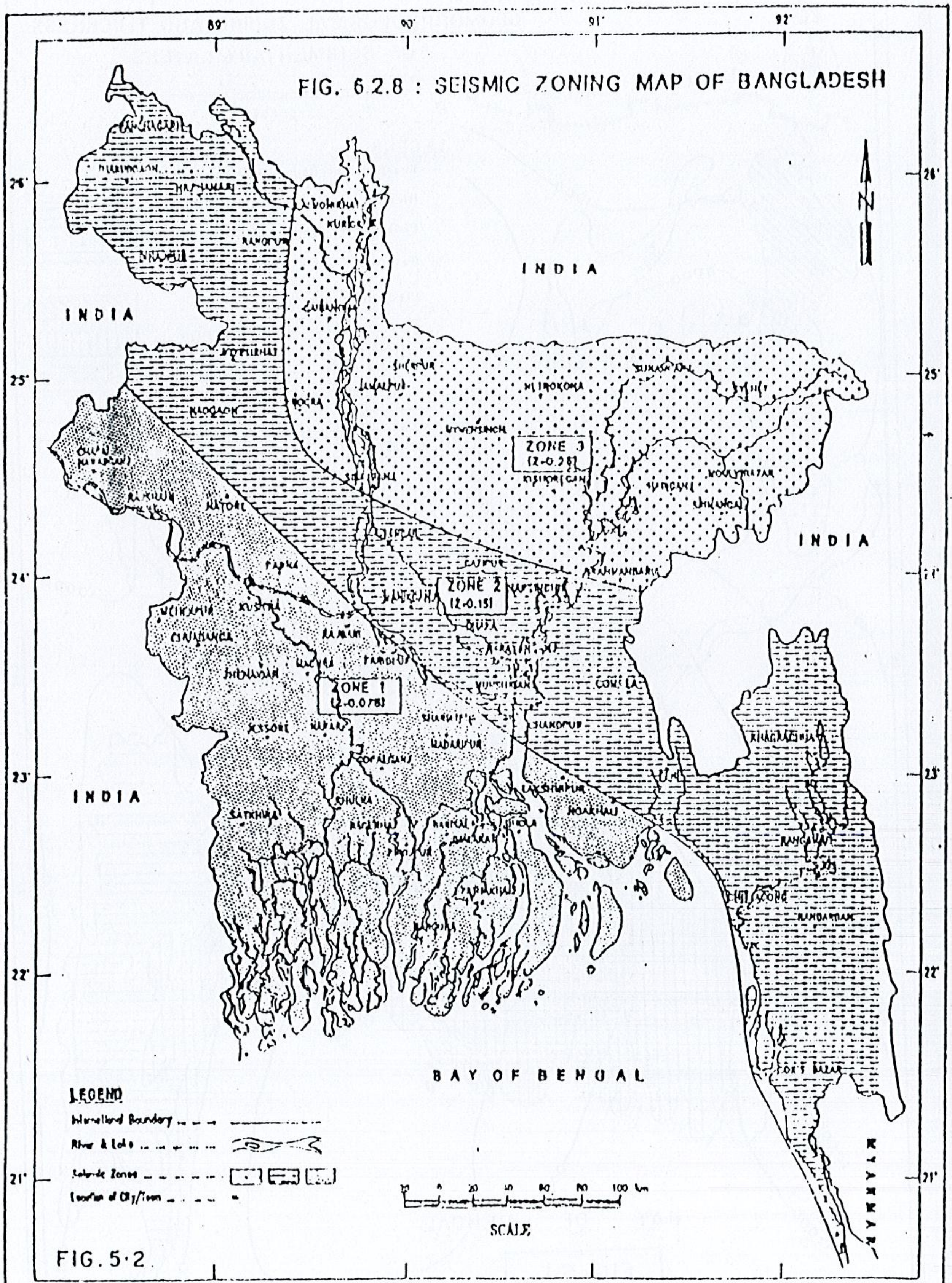


FIG. 6.2.8 : SEISMIC ZONING MAP OF BANGLADESH

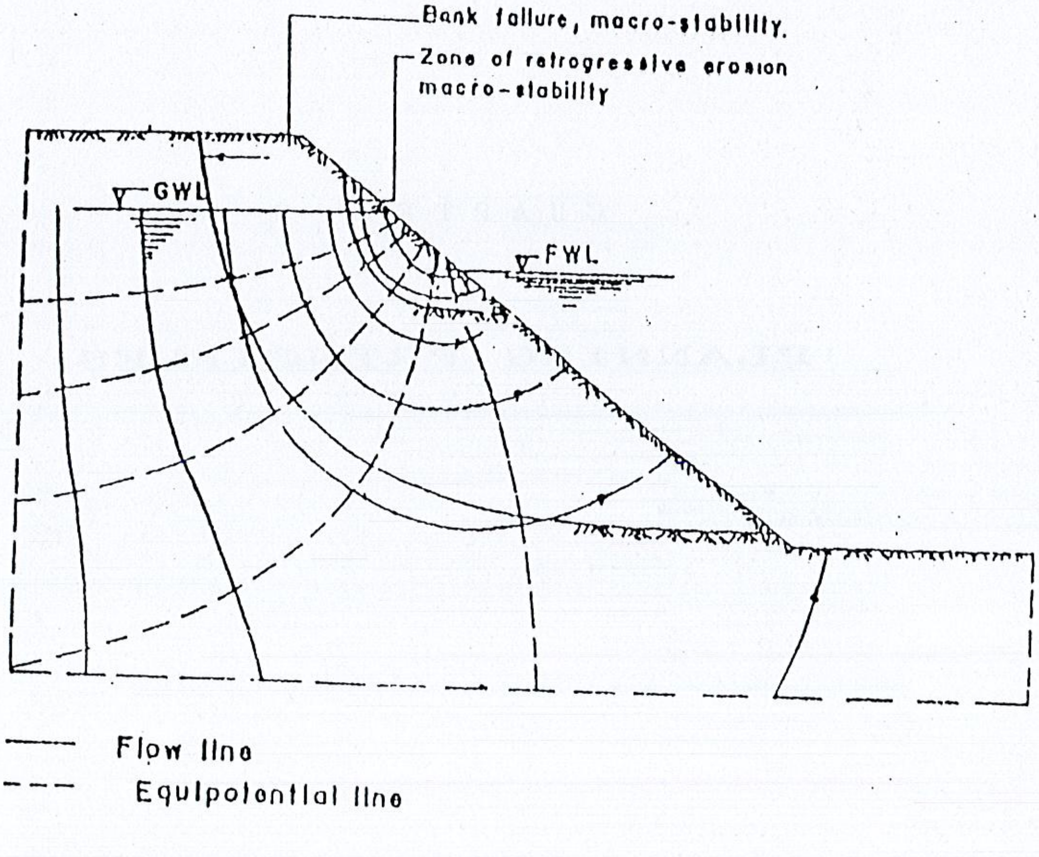
FIG. 5·2

BOUNDARY CONDITION

Macro- stability and micro-stability
Soil type : Medium dense to dense sand
Ground water level > Free water level

FAILURE PHENOMENON

Reduction effective stress
Loss of micro-stability near top +
Loss of macro-stability inside =
Combined phenomenon



MACRO AND MICRO – STABILITY IN SINGLE LAYERED SOIL | FIGURE – 5.3

GUIDE TO PLANNING AND DESIGN OF
RIVER TRAINING AND BANK PROTECTION WORKS

CHAPTER 6

PLANNING PRINCIPLES

CHAPTER 6

6.0 PLANNING PRINCIPLES

6.1 General Criteria

The main function of a bank protection work is to provide a substantial interface between the water flow and the containing ground. To achieve this the improvement must satisfy the following basic requirements.

- **Stability :** The work must be capable of supporting the imposed loads, to stabilize the underlying soil and to prevent erosion.

- **Flexibility :** The ability to absorb settlement deformations without impairment of its other functions.

- **Durability :** The structure should remain effective for the duration of its required design life at least.

- Maintenance : The design should allow maintenance including the repair of local damage and the replacement of deteriorated materials. The elements which require periodic maintenance should be easily accesible for inspection and replacement.

- Safety : During design consideration must be given to eliminating potential risk to the labour force and the public. All factors relating to safety should be incorporated, including consideration of all possible activities that may be taking place on and around the site, whether authorized or not.

- Environmentally acceptable : The works will be part of the environment and of the ecological system. To satisfy this requirement the project should not be designed with purely technical considerations in mind.

- Cost : The project will need to fulfill all the functional requirements while staying within budget for both construction and maintenance.

These requirements are achieved through an initial determinist design and a subsequent probabilistic verification.

6.2 Physical Model Test

Finally the different parameters of the selected river traini works may have to be determined and verified by conducting physical model test, preferably at the RRI, Faridpur.

The governing factor for the selection of the model scale willl the availability of space and hydraulic facilities at RRI and t possible effect due to surface tension at the time of fl measurement (which should be avoided). Depending on the abo condition a distorted scale may have to be chosen for the model.

Besides observation of General River Morphology, observation of t movement of the materials along the sides and bed of the river wi be of paramount importance, as such the model will be construct on a movable bed.

The relationship of dimensional and hydraulic quantities betwe model and prototype which is based on Froudian laws of similitu and is given below:

Velocity Ratio; $V_r = V_m/V_p = H_r^{1/2}$

Discharge Ratio, $Q_r = Q_m/Q_p = L_r H_r^{3/2}$

Time Ratio, $T_r = T_m/T_p = L_r/H_r^{1/2}$

Where, 'm' & 'p' represent respectively model and prototype. ' refers to ratio. L_r & H_r are Horizontal scale ratio and Vertic scale ratio respectively i.e. $L_r = L_m/L_p$ and $H_r = H_m/H_p$.

6.3 Bank Revetment

6.3.1 General

As per definition a revetment is a type of protection of a river bank or an 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 to be designed to protect and to stabilize the slope that may be subjected to action by waves and water current.

6.3.2 Selecting the Type of Revetment

There are various types of revetment and many possible combinations between them are also possible. A revetment consists usually of three components : cover layer, filter layer and toe protection. Its principal function is to provide a stable interface between the river and the bank line. Apart from the criteria mentioned in article 6.1, the selection of type of revetment for a particular site should also be based on the following factors :

. Availability of materials

. Durability of materials

. Easy construction and repair methods in case of local damage

. Possibility of monitoring and inspection of local damages

6.3.3 Dimensioning of the Revetment

Regarding the geometrical dimensions, a distinction is to be made between a revetment placed on an embankment and a revetment on the river bank. In the later case the alignment of the revetment has to follow the run of the river bank which is to be graded to required slope by excavation, filling, dredging and/or hydraulic backfill. Its crest and the elevation of the flood plain are the same.

In case the revetment has to protect the river side of an embankment being an artificial bank the alignment has to be determined in such a way that high current attack is avoided, however, often influenced by the topography and the infrastructure. The crest of the revetment is as high as the elevation of the embankment, the level of which shall be in case of permanent structures sufficient above the high water level of 100 years return period taking into account possible wave run-up. Though overtopping is to be avoided the land side slope of the embankment must be safe against slope failure and against erosion in case of overtopping in extreme flood events.

The crest width should be sufficient to provide a road for supervision and transportation of materials for maintenance and emergency works. Pavement of the road will protect the embankment from damage. The type of pavement and the width of the road depends on the kind of traffic (light or heavy).

The slopes to be applied to the revetment of banks and embankment depend on the quality of the subsoil, the flow of ground water and the selected type of revetment. Analysis of slope stability is required in each case.

Moreover, a berm on the river side of the embankment should be provided to facilitate the execution of construction and maintenance works, to improve the stability and/or to a have a good transition between different types of revetment. Its level should be above such a water level allowing construction and maintenance in the dry during a certain period of time annually. The width of the berm depends on stability analysis and dimension of construction and maintenance equipment.

6.4 Groyne

6.4.1 General

Groynes vary considerably not only in their construction and appearance, but also in their action on stream flow. According to the method and materials of construction they can be permeable or impermeable. Permeable groynes slow down the current whereas impermeable groynes deflect the current.

6.4.2 Selecting the Type of Groyne

A first step in the planning process of a bank protection with groyne is the selection of the type of groyne :

- a single or a series of groynes;

- a permeable or a impermeable groyne, or,

- a submerged or a non-submerged groyne.

Groyne can be used single or in series. Their use in series arises if a long reach is to be protected and depends also on the flow, deflecting or repelling. Furthermore, the flow pattern between groynes built in series can be influenced by an optimized layout plan as are the location and of scour. Permeable groynes have an advantage if placed in a series, because of several permeable groynes, the reduction of the flow velocity near the bank will be enhanced by properly selected spacing of the groynes. Impermeable groynes do not have this advantage.

In a standard layout of a series of groynes all the groynes are similar regarding length, cross-section, orientation and the shape of the head of the groyne. However, often the most upstream groyne is attacked by th flow stronger than the other downstream groynes, and in that cxe the design of that groyne should be adjusted. In principle a selection from different options can be made for this adjustment : either the length of the groyne can be reduced, or the orientation can be changed or the top layers can be stronger than those of the other downstream groynes.

Single groyne protect the bank downstream of the groyne over a relatively short distance against bank erosion. Due to their sensitivity to change directions of flow attack, single groynes are not recommended for general application.

To have a maximum effectivity on the flow during the maximum flood the groyne should not be submerged. However, if the maximum effectivity of the groyne is required during lower discharge, a submerged groyne can be cost effective, for example to improve the navigation depth during low discharge.

For special situations a combination of the mentioned types groyne is applied in one groyne, for example a submerged groy head and a not submerged shank of the groyne, or a permeable groy head connected to a non-permeable shank to the embankment.

6.4.3 Length and Spacing of Groyne

The length of groyne is governed by the shape of the cross-secti of the river and the extent of protection of the bank required. T latter leads obviously to a correlation with the spacing of groyn when constructed in series. In case a single groyne is planned, length must be sufficient to protect the bank against erosion the required area.

The width of the cross-section of the channel at bankfull st should be blocked partially by a groyne to deflect the flow fr existing bank, if no more bank erosion is allowed or acceptabl Generally the length of the groyne is selected between 10% maximum 50% of that width. The length is often limited by the wi not to construct the groyne in the deepest part of the chann close to the thalweg, because the cost per linear metre f construction in deep water are high. Along outer bends often seri of relatively short groynes are built, because of the great wat depths along the outer bend (bend scour and local scour).

In a mobile river, the selection of the length should be based a morphologic prediction of future development of the chann cross-section including future bank erosion and the development the thalweg of the channel. The basic length of the groyne of abo 5 to 15% of the channel width is increased by the width of t attached chars in front of the embankment. The construction of groyne on the attached chars is relatively cheap, because of t small construction height and since most of the construction can made in the dry.

Another approach can be applied if some bank erosion is acceptabl In such a case, the head is designed at the existing bankline a length of the groyne includes the expected embankment. The n embakment is retreated and designed with some safety with regard the extreme future bank line.

Experiences with groynes in Indian rivers show that a blockage c
30% of the channel width at bankfull stage can be used as
guideline (Varshney et al., and Jogleker, 1971).

The ratio between length and spacing between the groynes is one c
the most important factors of the effectiveness of a groyne field
In case groynes are spaced too far apart, the current may return t
the bank before the following groyne in the system is able t
influence the flow direction, resulting possibly in bank erosion c
even loss of the next downstream groyne. If groynes are spaced to
closely together, the groyne system would be too costly for it
effectivity.

In general practice is to relate the spacing of groynes to thei
length. The spacing depends not only on the length of the groyne
but also slightly on the orientation to the flow velocity, the ban
curvature, and purpose of the groynes.

General practice is a spacing of about 2 to 3 times the length o
groynes. This rule includes some safety for the bank erosion. I
favourable conditions and without additional safety a single groyn
can protect 4 to 5 times the length of the groyne. An India
guideline suggests a spacing of the groynes by 0.1 to 0.15 of th
meander length of an outflanking channel. If the meander length i
15 to 30 times the channel width and length of the groyne is 30% o
the channel width, then the spacing is 0.6 to 1.5 times the lengt
of the groyne.

For impermeable and permeable, non-submerged groynes along a
straight bank, a spacing of 2.5 to 3 times the length of the groyne
is recommended for bank protection. This recommendation includes
some safety.

On concave outer banks, groynes are to be placed closer together
than on straight banks and the design ratio should be reduced to 2
to 2.5. On a convex bank the spacing between the groynes can be
slightly more than the recommended spacing along a straight bank
(as a first indication a design ratio of 3 to 3.5)

The ratio of spacing to the length of groynes required for bank protection only is less than that required for navigation channels.

The length of the most upstream groyne of a series of groynes should be less than the standard length of the downstream groynes because of the strong attack on this upstream groyne. The reduction in length depends on each individual situation and no general recommendation can be given.

For all special situations it is recommended to perform physical model tests to determine both the optimal length of the groynes and their optimal spacing. In the design of the physical model the actual and future morphological and hydraulic boundary conditions should be taken into account as accurately as possible.

6.4.5 Shapes of the Groyne Heads

Different riverside ends of the groynes, called groyne heads, are being applied, mainly to reduce the scour of the river bed.

The main types of groyne heads of impermeable and not-submerged groynes are described in the following :

Bell head

A strongly protected head is also called a bell-head. The separation of the flow can shift easily along this head and consequently the direction of the vortex street downstream of the seperation point. A bell head is often used if the direction of the approach flow is more or less fixed.

T-head and L-head

Straight groynes with a rectangular vane at their heads are T-head or L-head groynes, they give more guidance to the flow and decrease the separation of the flow at the downstream side of the vane. The vane is oriented in such a way that it guides the flow. A T-head vane has two wings with an equal length. If the groyne blocks a relatively high percentage of the cross section (for example than 30%) a T-head reduces the risk of erosion at the opposite ba downstream of the groyne. A long upstream wing can reduce the attack on the shank of the groyne and allow a reduction of t strength of the top layer of the shank. The wings have in gener a strong top layer. In case there is only one wing they are call L-head groynes. The effect of the latter ones can be to have larg sediment deposits between the groynes, to provide a bett protection to the bank and to have less scouring at their head. head or L-head groynes are often built when the direction of t approach flow upstream of the groynes can change due to t morphological development of the river channels.

Hockey-stick head

Groynes bent like a hockey stick at their head are known as hock shaped groynes, which is a combination of a short L-head with bell head. Along the upstream side of the hockey stick the fl seperation point can shift but the alignment to the flow reduc the intensity of the vortex street and also reduces the local sco depth. This shifting of the flow seperation point is generally n attractive because the flow pattern downstream of the groyne is n well defined. A hockey-stick is applied if a very strong attack the head is expected, stronger than in case of a bell head, and a small reduction of this attack allows the selection of a le strong top layer of the head.

Since no general rules are available for determining the mos convenient shape of the groyne head as well as other dimensior depending on a number of different design parameters, it is in mar cases advisable to undertake physical model tests.

6.4.6 Cross section

A typical cross-section of a groyne depends on many considerations one of the most important ones being whether the groyne should b conceived as permeable or impermeable.

6.4.6.1 Permeable Groyne

Permeable groynes are generally constructed of timber, steel or reinforced concrete piles driven or sunk into the river bed in one or several rows. The individual vertical piles are mainly subject to horizontal loads, either on their total length by flow and wave attack or at changing heights due to floating debris, etc. In order to absorb the decisive horizotal loads, static and dynamic, it is advisable to have at least two rows of piles braced to each other by traverses and diagonals, the dimensions of which should be designed in accordance with standard rules of structural engineering. The spacing between the piles should be at least one diameter, hence the minimum permeability would be 50% of the area. For a standard design a maximum permeability of 70% is recommended, because of the effect on the flow field. Two parallel rows of piles can be placed in two different positions relative to each other. No recommendation is given for a certain arrangement.

The driving depth of the piles depends mainly on the subsoil conditions, the local scour hole and the pile characteristics. A falling apron reduces the local scour between the piles and reduces the depth of piles driven into the soil. The height of the piles is normally 1 meter above the maximum design water level. In special cases the height of the piles can be reduced to below this level which would result in a submerged permeable groyne during high floods. To drive inclined piles to absorb the horizontal loads is much more expensive than to drive vertical piles and therefore only vertical piles are envisaged for a standard design in Bangladesh.

The dimensions of the piles are determined according to the methods applied in structural and soil mechanical engineering and the design conditions have to be carefully assessed for each individual case.

To prevent local scouring close to the pile row a bed protection or a light falling apron of rip-rap or boulders is recommended. By increasing the permeability of the pile row towards the head of the groyne the maximum depth of the local scour hole can be significantly reduced, this scour reduction being the main reason for applying permeable groynes.

6.4.6.2 Impermeable groyne

Standard cross sections of the shank of an impermeable groyne have a trapezoidal shape: two sides slopes and a horizontal top. The top of an impermeable groyne should have a minimum width of about 3 m, so that a truck can pass for maintenance. If the sides of the shank have a protective top layer there slope should be designed as steep as the stability of the top layer allows to minimise the area of the top layer. This results in slopes of 1:2 to 1:3 at the shanks. A light falling apron or bed protection is recommended at the upstream side of the shank of the groyne.

The side slope around the head of the groyne is same as the side slope of the shank if no important scouring is expected. If a deep scour hole is predicted, then the depth of the scour hole can be reduced by flattening the side slopes of the head of the groyne to about 1:3 to 1:5 at the head of an impermeable earth built groyne.

Due to different construction methods above and below the low water level, the designs of the top layers of inclined slopes are often different above and below design low water level. The top layer below low water level must have a rather flat slope (1:3 to 1:4) for stability reasons or to reduce the scour depth, whereas the slope above design low water level may be steeper to save space and construction material. The transition from one top layer design to another is often less resistant to flow attack than the actual top layers, Therefore, a berm with a width of at least 1,5 m is designed at the transition.

The radius of the groyne head is determined by the effect of the head on the deflection of the flow, and the minimum radius may also be determined by the size of the top layer elements. For example the spaces between concrete blocks of a top layer should be limited to a certain maximum value to safegaurd their stability and interlocking properties.

The crest of an impermeable groyne can be designed at standard high water level with a gentle slope of 1 : 100 towards the river along the axis of the groyne. This gentle slope assures a gradual increase of the submerged length of the crest during the rising limb of an extreme flood, and local high flow velocities during the flood above the crest are prevented. The crest of a permeable groyne may be either horizontal or have the same gentle slope as l case of an impermeable groyne. In a special case, e.g. for dee water and/or strong flow and wave attack, a steeper slope of th crest may also be appropriate.

It must be stressed that the above mentioned slope are not onl depending on the hydraulic loads, but also on the constructio material of the core and protection of the slopes. In any cas detailed stability investigations are required in order t establish the required slope, as well as estimations of settlemen of the earth built dam depending mainly on the subsoil condition

The existing river bed around a groyne should be protected fro scouring by a bed protection or a falling apron. A permanent be protection has a length of about 1.5 times the scour depth at th downstream edge of the bed protection. In practice this leng proved to be sufficient to guarantee the stability of the groyn

6.4.7 Orientation of Groyne

Groynes facing downstream are not suitable for bank protecti surfaces, since the current may attack the root of the ne downstream groyne endangering not only the root and the surroundi bank area, but the whole groyne itself. Groynes placed normal the flow may protect only a small area. However, groynes faci upstream deflect the flow away from the bank, and they are able t protect bank areas upstream and downstream of themselves. Th deflecting groynes seem to be best suited for bank protection sedimentation purposes. Design guidelines recommened angles of 10 to 120° to the flow, however, also the form of the bank, that mea concave or convex, must be taken into consideration. In straig reaches groynes may be pointed upstream as much as 105° to 110 Typical design of a groyne is shown in fig.6.1.

6.5 Guide Bundh

6.5.1 Geometrical Shape

The Guide Bundh can be named according to the shape of the curved head, namely, straight and elliptical guide bundhs with circular or multi radii curved head (fig.6.2). Elliptical guide bundhs have been found more suitable in case of wide flood plain rivers as compared to straight guide bundhs. Due to gradual change in curvature in the former case, the flow hugs the guide bundhs all along its length as against seperation of flow occurring in case of straight guide bundhs after the curved head which leads to obliquity of flow. Elliptical guide bundhs have also been found to provide better control on development and extension of meander loop towards the approach embankment. Any other shape warranted by site conditions and supported by hydraulic model studies may be adopted.

6.5.2 Length of Guide Bundh

The length of the guide bundh on the upstream should normally be kept as 1.0 L to 1.5 L. In order to avoid heavy river action on the guide bundh, it is desirable that obliquity of flow to the river axis should not be more than 30° (fig.6.3).

For wide alluvial belt the length of guide bundhs should be decided from two important considerations, namely, the maximum obliquity of the current and the permissible limit to which the main channel of the river can be allowed to flow near the approach embankment in the event of the river developing excessive embankment behind the training works. The radious of the worst possible loop should be ascertained from the data of the acute loops formed by the river during the past. In case of river where no past surveys are available, the radious of the worst loop can be determined by dividing the radious of the average loop worked out from the available surveys of the river by 2.5 for rivers having a maximum discharge upto 5000 m^3/s and by 2.0 for a maximum discharge above 5000 m^3/s.

The length of the ... the down stream side should be kept as 0.25 L t

6.5.3 Curv ... de Bundh

The func ... upstream guide bundh is to guide the fl ... to the structure keeping the end spans ... the radius of the curved head of guide bundh should be kept as small as possible consistent with the proper functioning of the guide bundh. A radius of the curved head equal to 0.4 to 0.6 times the width of the barrage between the abutments have usually been found to give satisfactory results, the lower and upper limits of the radius being 150 and 600 m, unless indicated otherwise by model studies.

Radius of curved tail normally ranges from 0.3 to 0.5 times the radius of curved head.

The angle of sweep of curved head ranges from 120° to 145° according to the river curvature. For curved tail it usually varies from 45° to 60°. A typical layout of guide bundh is shown in fig.6.3.

In case of elliptical guide bundhs, the elliptical curve is provided upto the quadrant of ellipse and is followed by multi-radii or single radius circular curve. In case of multi-radii curved head the larger radius adjacent to apex of ellipse is generally kept 0.3 to 0.5 times the radius of the curved head for circular guide bundhs, with angle of sweep varying from 45° to 60° and the smaller radius equivalent to 0.25 times the radius of curved head for circular guide bundhs with sweep angle of 30° to 40° (fig.6.4). The shape should be finalised on the basis of model studies.

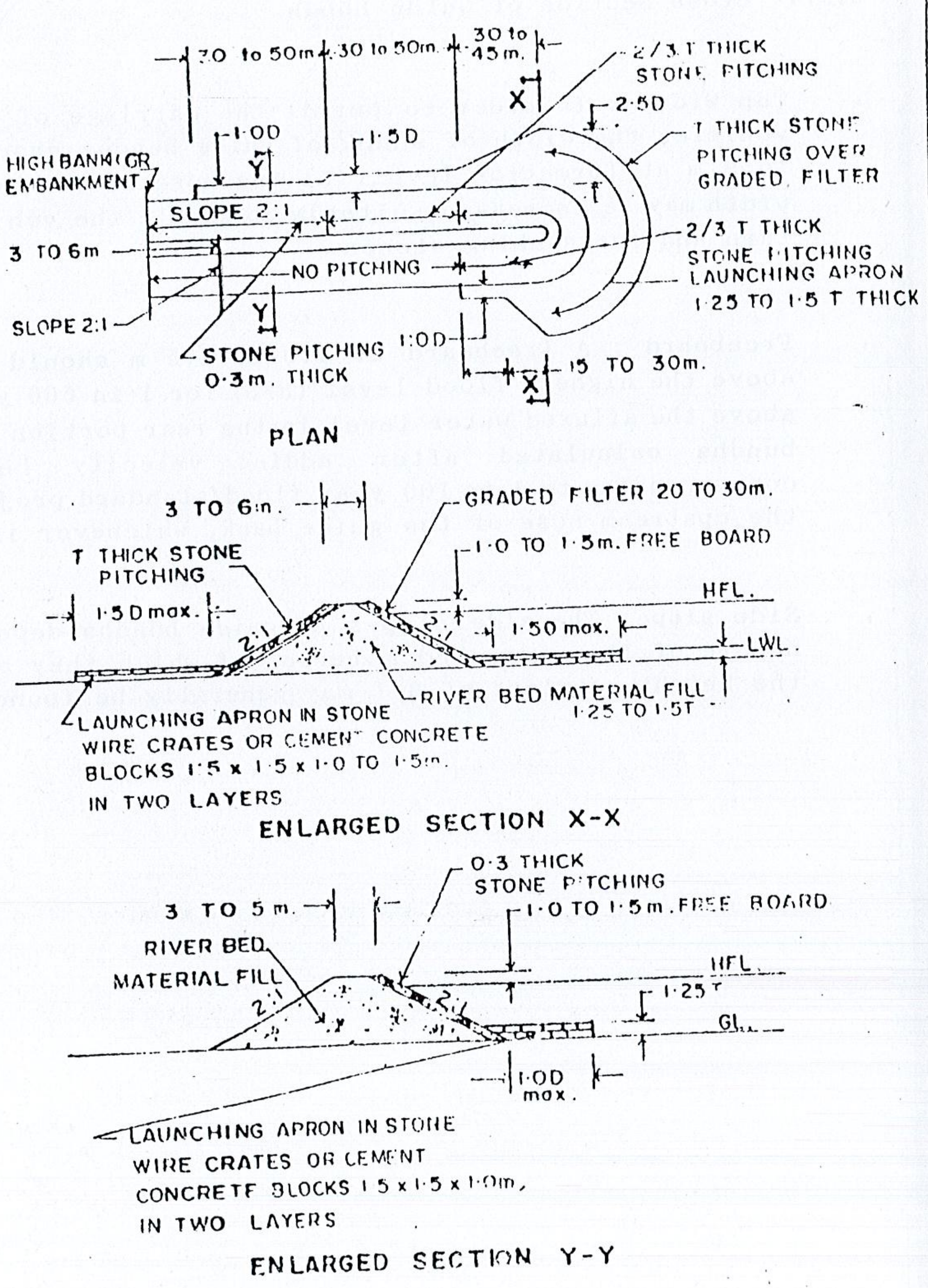
The layout, length and radii of curved head of the guide bundhs should generally be decided on the basis of hydraulic model studies taking into consideration the guide lines indicated abobe.

6.6.4 Cross Section of Guide Bundh

- Top Width -: In order to permit the carriage of material and vehicles the width of shank of guide bundhs should be kept 6 to 9 m at formation level. At the nose of guide bundhs, the width may be increased suitably to enable the vehicles to take turn and for staking stones.

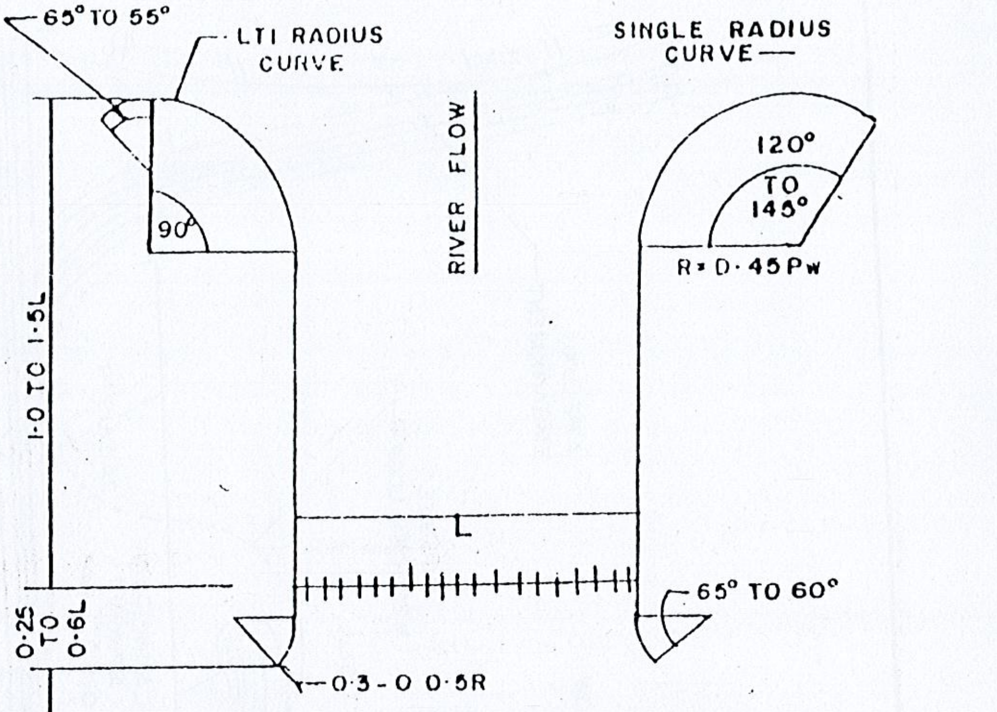
- Freeboard : A freeboard of 1.0 to 1.5 m should be provided above the highest flood level (HFL) for 1 in 500 year flood or above the afluxed water level in the rear portion of the guide bundhs calculated after adding velocity head to HFL corresponding to 1 in 100 year flood/standard project flood at the upstream nose of the guide bank, whichever is higher.

- Side slope: The side slopes of guide bundhs depend upon the nature of the river bed material of which they are made and the height. A slope of 2:1 may generally be found adequate.

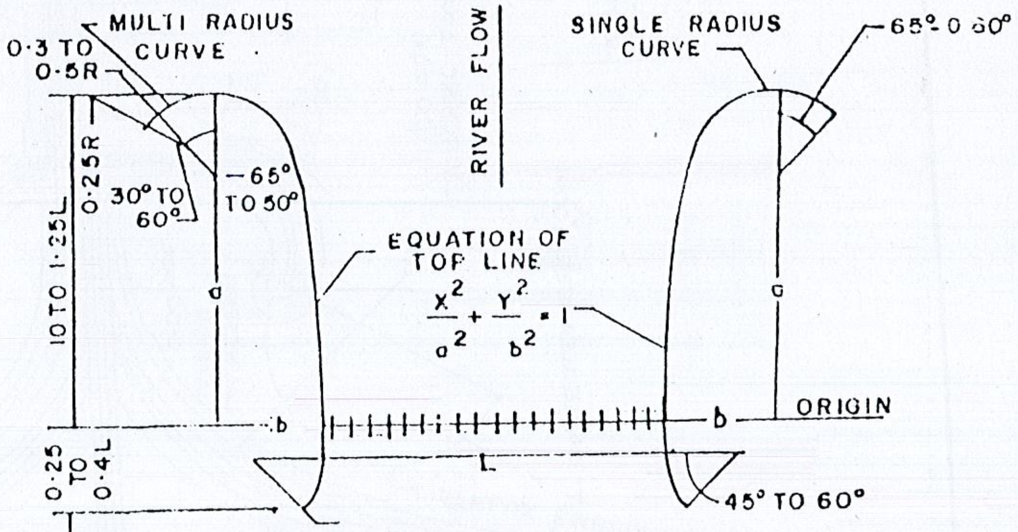


TYPICAL DESIGN OF SPUR

FIG. 6.1



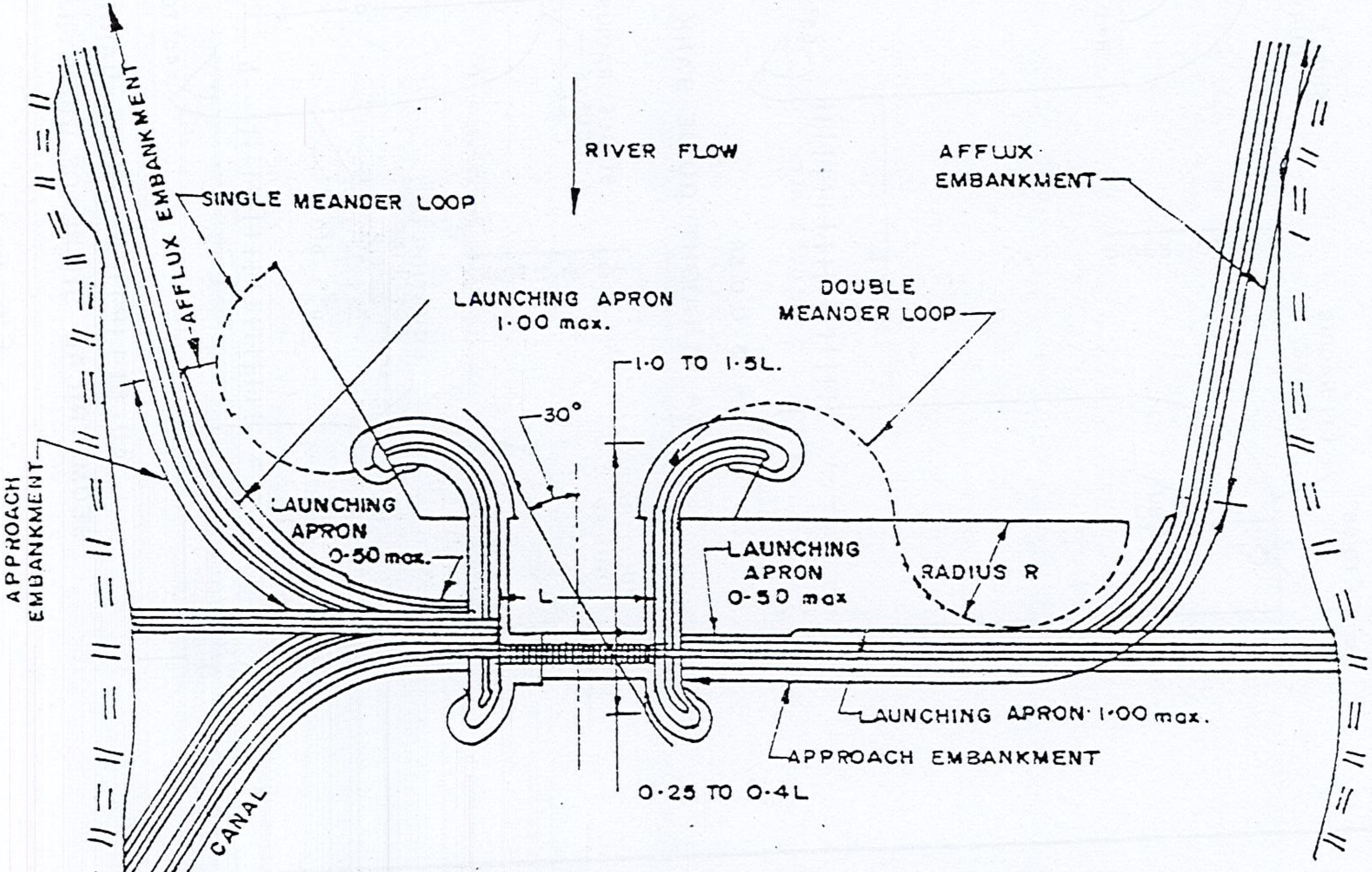
3A STRAIGHT GUIDE BANK



3B ELLIPTICAL GUIDE BANK

GEOMETRICAL SHAPE OF GUIDE BANKS

FIG. 6·2



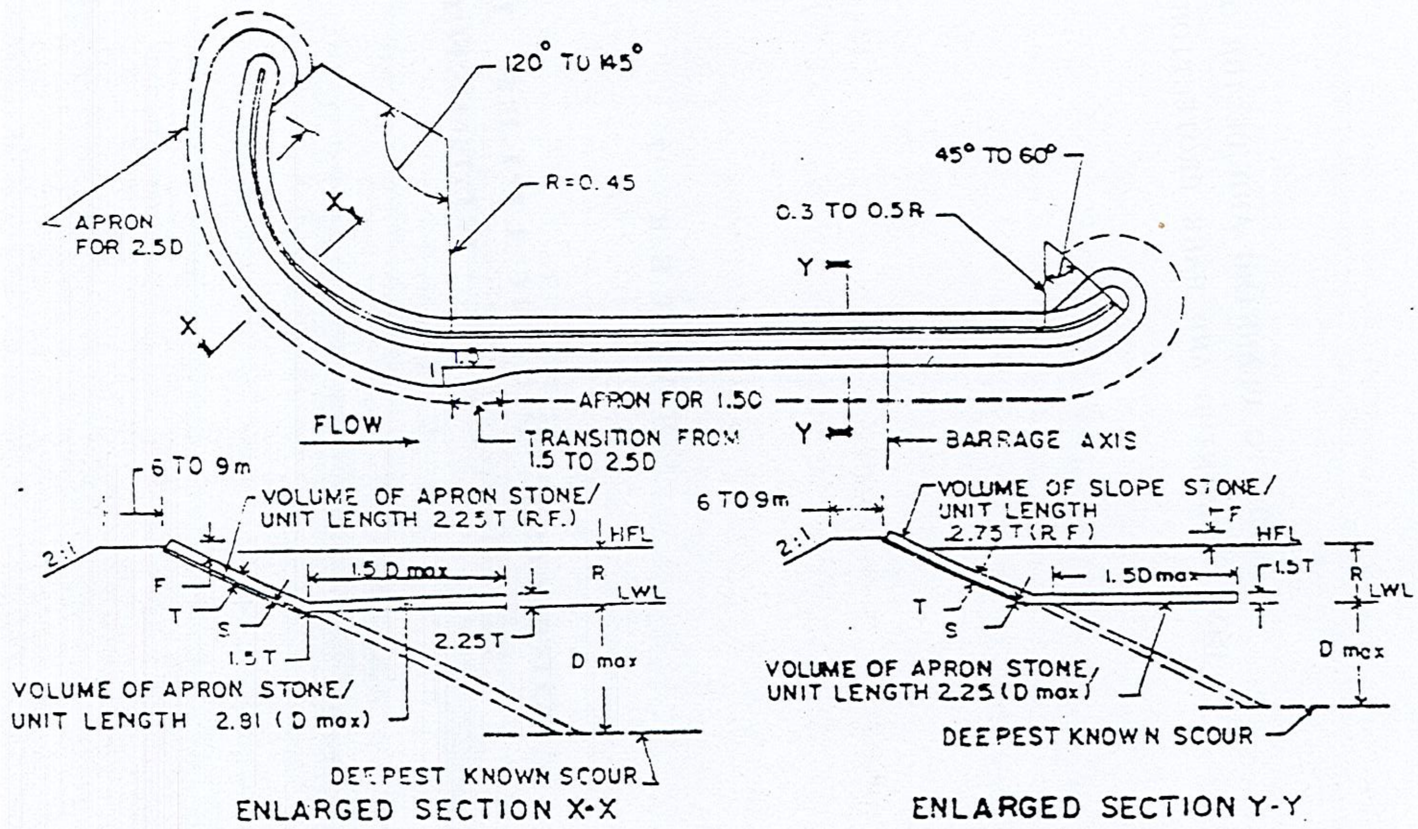
TYPICAL LAYOUT OF RIVER TRAINNG WORK

FIG. 6-3

TYPICAL LAYOUT OF RIVER TRAINNG WORK

TYPICAL LAYOUT OF RIVER TRAINNG WORK

FIG. 6-3



LEGEND:

F Free board

R Rise of flood above low water level

D max Deppth of scour for calculation of apron stone

T Thickness of slope stone

S Thickness of soling (Filter)

DETAILS OF GUIDE BANK

FIG. 6-4

GUIDE TO PLANNING AND DESIGN OF
RIVER TRAINING AND BANK PROTECTION WORKS

CHAPTER 7

DESIGN PRINCIPLES FOR STRUCTURAL ELEMENTS

CHAPTER 7

7.0 DESIGN PRINCIPLES FOR STRUCTURAL ELEMENTS

7.1 General

Whatever may be the type of structure used in river training works e.g. bank revetment, groyne or guide bundh, the protection works can be divided into two main parts i.e. protection of bank itself and protection of toe of the bank. The protection of bank is done by revetment using selected materials and the protection of toe is done by launching apron or driven piles.

The following general criteria should be considered during design of any protection works :

- the revetment does not slide under freqently occurring hydraulic loads
- the revetment including filter layers and subsoil must be in equilibrium as a whole
- the component of the weight of the revetment normal to its face should be greater than the uplift pressure caused by water
- the surface particles of revetment should have enough resistance against wave and current attack
- the toe of revetment shall be stable against probable maximum scour in the river bed,

The structural elements of the protection works need to be selected to fulfil the above criteria are as follows :

- size and thickness of revetment materials in open type revetment
- thickness of revetment in close type revetment

.. thickness and gradation of riprap

. thickness and gradation of granular filter

. type of fibre filter

. dimension of launching apron

. depth of pile used as toe protection.

7.2 Stability of Revetments under Current Attack

7.2.1 Loose Units

A number of equations relating the size of revetment material to the velocity impinging on it, has been developed. The most common of these have been presented here and a recommendation is made for the equation that is considered to be the most suitable.

1. Isbash [Ref.- 9] $$W = \frac{4.1 \times 10^{-5} S_s V^5}{(S_s-1)^3 . \cos^3\theta} \quad \text{(FPS unit)}$$

2. California State Highways (CSH) [Ref.- 5]

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

3. Neill [Ref.- 12]. This is in the form of a design graph (see fig. 7.1) for spherical stones (specific gravity = 2.65) and for bank slopes between horizontal and vertical, 1(V) : 2(H). The following equation has been fitted to the curve.

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

4. Pilarczyk [Ref.- 14]

$$D = \frac{V^2}{36\ g\ (S_s - 1)\ \Psi\ (h/D)^{1/6}} \qquad \text{(FPS or metric unit)}$$

5. PIANC [Ref.- 13]

$$D_n = \frac{0.7\ V^2}{g\ (S_s - 1)\ \cos\theta\ (1 - \tan^2\theta/\tan^2\Phi)^{0.5}}$$

(FPS or metric unit)

6. JMBA [Ref.- 10]

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

(FPS or metric unit)

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/s^2 or m/s^2)

The definition of velocity is not fixed. The Neill equation is in terms of the velocity against the stone, in other case it is the mean velocity in the adjacent channel or the velocity at any specified distance from the bank.

The PIANC equation which is stated to be a form of Isbash equation gives much lower size of revetment materials compared to other three equations. The equations suggested by Pilarczyk and JMBA both considered the depth factors for computation of size of revetment materials but the formar gives higher size compared to latter one. Neill equation and JMBA equation (taking depth factor h/D = 5) gives nearly equal size of revetment materials. Considering all these facts, it is recommended to use Neill and JMBA equations for computation of revetment size against velocity.

7.2.2 Stone Gabions and Mattresses

(1) The following formula to determine the grain diameter of stones for filling of gabions and mattresses is given by PIANC (1987a

$$d_{n50} = h * [\frac{U_{cr}}{B\sqrt{K\psi_{cr} g \Delta_m h}}]^{2.5}$$

where,

d_{n50} = grain diameter (m)

= $(W_{50}/\rho_s)^{1/3}$

h = water depth (m)

k' = slope reduction factor

u_{cr} = critical velocity (m/sec)

= $[1 - (\mathrm{Sin}^2 a)/(\mathrm{Sin}^2 \xi_s)]$

ξ_s = angle of internal friction of material

ψ_{cr} = Shields parameter

= 0.1 in case of initiation of movement

B_o = cofficient of flow conditions

= 5 to 6 for major turbulent flow and other bends of river

= 7 to 8 normal turbulence of rivers

= 8 to 10 for uniform flow and minor turbuleance

Δ_s = relative density (-)

= $(\rho_s - \rho_w)/\rho_w$

ρ_s = density of protection material (kg/m^3)

ρ_w = density of water (kg/m^3)

The thickness of the mattress can be related to the stone size. According to PIANC (1987a) it is sufficient to use two layers of stones in a mattress.

$$t_m = 1.8\ d_{m50}$$

in which,

t_m = thickness of mattress (m)

(2) The following equation is again derived from the general Pilarczyk formula and valid for wire mattresses :

$$D_n = \frac{\bar{u}^2}{\Delta_m \, 2g} * \frac{0.03}{\Psi_{cr}} * K_h * K_s$$

where,

$\bar{u}$ = current velocity (m/sec)

Ψ_{cr} = Shields parameter (-)

K_h = Depth factor (-)

$$= \frac{2}{[\log(6h/D)]^2}$$

h = water depth (m)

$D = D_n$ = nominal thickness of protection unit (m)

Ks = Slope factor (-)

$$K_s = \cos\alpha \sqrt{\frac{1 - \tan^2\alpha}{\tan^2\xi_s}} - \sqrt{1 - \frac{\sin^2\alpha}{\sin^2\xi_s}}$$

In which,

α = angle of slope (degree)

ξ_s = angle of internal friction between cover layer and sub layer (degree)

Since the stones are retained by the wire mesh, the Shields paramater can be increased from Ψ_{cr} = 0.03 to Ψ_{cr} = 0.06 to 0.10 in case of initiation of movement.

For the estimation of thickness of a geotextile mattress the following equation can be applied :

$$d \geq \frac{\Phi \bar{u}^2}{\Delta_m 2 g} * K_h * K_s$$

where,

d	=	thickness of mattress	(m)
Φ	=	0.8 for continuous protection	(-)
	=	1.0 for exposed edges	(-)
K_s	=	slope factor	(-)
K_h	=	depth factor	(-)

7.2.3 Grouted Stones and Open Stone Asphalt

Since current attack is normally not critical for grouted stones and open stone asphalt, no formula is available.

7.3 Stability of Revetment under Wave Attack

The principal factor affecting the design of slope protection is wave action. A riprap layer must be designed to protect the individual rock particles from displacement by wave forces. Riprap protection against wave action is generally satisfactory for protecting slope from other attacks.

7.3.1 Loose Units

Various methods and equations are available to determine the size of revetment material required to resist wave forces. Some of these formulae are as follows :

(1) CHP [Ref.- 4] : $$W = \frac{0.00231\, H^3 \rho\, \mathrm{Cosec}^3(70^\circ - \theta)}{\Delta^3}$$

(2) Hudson : $$W = \frac{H^3\, \rho\, \tan\theta}{k\, \Delta^3}$$

(3) Iribarren : $$W = \frac{f\, H^3\, \rho}{\Delta^3(\cos\theta - \sin\theta)^3}$$

(4) Pilarczyk [Ref.- 13] : $$D = \frac{H_s}{(S_s - 1)} \cdot \frac{1}{\phi} \cdot \frac{E^{1/2}}{\cos\theta}$$

Where,

W = Weight of revetment material

D = Cubic dimension

H_s = Significant wave height

ϕ = Strength coefficient, 3 for cubes and 2 for randomly dumped cubes

θ = Bank slope angle

E = Wave breaking parameter

= $1.25\ T/H_s^{0.5} \tan\theta$

T = Wave period (sec)

f = A coefficient related to the amplitude of the wave and the depth and angle of the slope.

ρ = Density of revetment material

Δ = Relative density of revetment material

= $(\rho - \rho_w)/\rho_w$

ρ_w = Density of water

k = A coefficient varying from 3.2 for smooth quarry stone to 10 for tetrapods

S_s = Specific gravity of revetment material

(5) The empirical form of Hudson's equation in fps unit with k=3.2, is

$$W_{50} = \frac{19.5\ G_s H_s^3}{(G_s - 1)^3 \cot\theta}$$

The solution of equation is shown graphically in fig 7.2, and may be used for preliminary estimation of sizing riprap rock on wave consideration.

Among the above formulae, the Pilarczyk equation is currently used in many big river training works. Hence, this equation may be used.

(6) Another form of Pilarczyk equation which computes the thickness of revetment (t) is expressed as follows :

$$t = \frac{G}{\cos\theta} * I_r^{1/2} * \frac{H}{S_s - 1}$$

Where,

G = Constant

= 0.19 to 0.26 for ungrouted concrete blocks

I_r = Irribarren Number

$= \tan\theta/(H_s/L_0)^{1/2}$

L_0 = Deep water wave length

$= gT^2/2\pi$

H = As mentioned above

S_s = As mentioned above

7.3.2 Gabion and Mattress

The gabion or mattress of the thickness d must be stable as a unit. The thickness can be related to the stone size D_n. In most cases it is sufficient to use two layers of stones in a mettress and

$$d = 1.8\ D_n$$

(1) The thickness d of the gabion/mattress as well as the nominal diameter D_n of the stone fill can be determined by means of the Pilarczyk formula :

$$d = \frac{H_s\ \xi_z^{\ 0.5}}{\Delta_m\ \Psi_u\ 2.25\ \mathrm{Cos}\ \alpha}$$

where,

ξ_z = breaker similarity index

Ψ_u = 2 to 3

Δ_m = relative density of the system

Δ = relative density of fill material
= $(1 - n)\ \Delta_m$

α = slope angle

an.

$$D_n = \frac{H_s \; \xi_z^{0.5}}{\Delta_m \; \psi_u \; 2.25 \; \cos \alpha}$$

in which,

$$\psi_u = 2 \text{ to } 2.5$$

$$\Delta_m = 1.65$$

(3) For the estimation of the thickness of the gabions/ mattress PIANC (1987a) gives the following equations :

$$t_m = \frac{H_s}{2\,(1-n)\;\Delta_m \;\cot \alpha} \qquad \text{for } \cot \alpha < 3.0$$

$$t_m = \frac{H_s}{4(1-n)\;\Delta_m\;(\cot \alpha)^{1/3}} \qquad \text{for } \cot \alpha >= 3.0$$

where,

t_m = thickness of mattress

H_s = significant wave height

n = porosity of stone = 0.4

However, these equations are only used for significant wave heights of less than 1.00 m.

7.3.3 Grouted Stones

In case of grouted stone used as cover layer on a slope revetment the proper execution of grouting is of outstanding importance. Care must be taken to ensure that the grout does not remain in the surface of the stone layer only or sags completely through the layers. Creating of a completely impermeable surface must be avoided because it may introduce extra lift forces.

(1) Pilarczyk recommends for the application of his general formula as follows :

$$D \geq \frac{H_s \, \xi_z^{\,b}}{\Delta_m \, \psi_u \, \phi \, \cos\alpha}$$

Where,

D = specific size or thickness of protection unit (m)

H_s = significant wave height (m)

Δ_s = relative density of a system unit (-)

Φ = stability factor (-)

ψ_u = system determined stability upgrading factor (-)

= 1.0 rip-rap as a reference

>= 1.0 for other revetment systems

= 1.05 for surface grouting (30% of voids)

= 1.50 for pattern grouting (60% of voids)

α = slope angle (degree)

ξ_z = breaker similarity index (-)

$= \tan\alpha * (H_s/L_0)^{-0.5}$

$$= \frac{1.25 * T * \tan\alpha}{(H_s)^{1/2}}$$

in which,

L_0 = wave length (m)

$$= \frac{g\,T^2}{2\,\pi}$$

T = average wave period (sec)

b = in the range of 1/2 to 2/3

(2) The following formula is given by PIANC (1987a) :

$$D_{n50} \geq \frac{H_s}{\Delta_m\,(K_D \cot\alpha)^{0.33}}$$

where,

K_D = damage coefficient as per CERC (1984)

Depending on the degree of grouting the damage coefficient may be upgraded by the following factors :

For surface grouting : 1.0 to 1.5

For pattern grouting : 5.0

Upgrading of K_D-values give reductions in the sizes of stones which are approximately

D_{n50} (grout) = 0.9 D_{n50} (loose) for surface grouting.

D_{n50} (grout) = 0.6 D_{n50} (loose) for pattern grouting.

7.3.4 Open Stone Asphalt

(1) The thickness of a cover layer of open stone asphalt exposed to wave attack can be determined by the following formula, PIANC (1987a) :

$$d_b = 0.75 * [\frac{27}{16} * \frac{1}{1-V^2} * (\frac{P}{\sigma_b})^4 * (\frac{S}{C})]^{1/5}$$

where,

d_b = thickness of cover layer (m)

σ_b = asphalt stress at failure (N/m²)

P = wave impact (N/m)

S = stiffness modulus of the asphalt (N/m²)

V = Poisson's ratio for asphalt (-)

C = modulus of subgrade reaction (N/m³)

The following thicknesses of open stone asphalt layers on rive[r] banks can be estimated as a guidance :

Small rivers :

d_b = 10 to 15 cm (in situ)

= 8 to 12 cm (prefabricatrd mattress)

Large rivers :

d_b = 15 to 25 cm (in situ)

= 15 cm (prefabricatrd mattress)

(2) The recommendations of Pilarczyk for the application of his general formula (see section 7.3.3) to determine the thickness of a stone asphalt layer are as follows :

b = 2/3

Ψ_u = 2.0 for open stone asphalt placed on geotextile on sand

= 2.5 for open stone asphalt placed on sand asphalt as sub layer. In this case the thickness of the system may be defined as the total thickness of both layers.

(3) Another formula furnished below has been used in Meghna River Bank Protection Study to compute the thickness of open stone asphalt layer :

$$\frac{H_s}{\Delta t} = \Psi * \frac{a}{E^{1/2}}$$

where,

γ	=	upgrading factor
α	=	stability factor
t	=	thickness of protection
H_s & E	=	as explained above

7.4 Thickness and Grading of Riprap

7.4.1 Riprap Thickness

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

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

- California Highway Division (1970) recommended that there should be at least two layers of overlapping stones so that slight loss of material does not cause massive failure.

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

- Spring (1903) recommended thickness of stone in inches for covering by rough, heavy and loose stone for pitching from low water upwards as shown in Table 7.2.

- the thickness of stone pitching and soling for permanent slopes required at head, body and tail of guide bank for river flowing in alluvial plains as recommended by Gales (1938) is tabulated in Table 7.3.

Inglis (1949) recommended following formula to compute thickness of stone required,

$$T = 0.06\ Q^{1/3}$$

Where,

T = Thickness of stone riprap in ft and

Q = Discharge in cfs

The Inglis formula apparently gives excessive thickness for dischsrge above 1.5 million cusecs.

**The thickness determined above should be increased by 50 percent when the riprap is placed under water to provide for uncertainties associated with this type of placement.

7.4.2 Riprap Grading

Though the exact size distribution of a riprap mixture is not critical, but it should form a smooth grading curve without a large spread between minimum and maximum sizes. Gradation of stones in riprap revetment affects the riprap's resistance to erosion. Stones should be reasonably well graded. Table 7.1 shows grading specifications (Ref.- 12) for three classes of riprap which have been considered suitable for a fairly wide range of stream flow situations.

The size obtained from figure 7.1 should be taken as median diameter (D_{50}), which means that 50% by weight of the mixture should be larger. All stones should be contained within the riprap layer thickness to provide maximum resistance against erosive forces. Oversize stones, even in isolated spots, may result in riprap failure by precluding mutual support and interlock between individual stones, causing large voids that expose filter and bedding materials, and creating excessive local turbulance that removes smaller size stones.

7.5 Design of Toe Protection

7.5.1 Toe Scour Estimation and Protection

Lack of protection of the toe of revetment against undermining is a frequent cause of failure of revetment. Accordingly, protection of the toe of revetment by suitable method is a must. This is true not only for riprap, but also for a wide variety of protection techniques. The scour is the result of several factors, including the factors mentioned below:

a. Change in cross-section in meandering channel after a bank is protected: The thalweg often moves towards the outer bank after the bank is protected. Channels with highly erodible bed and banks can experience significant scour along the toe of the new revetment.

b. Scour at high flows in meandering channel: Bed profile measurements have shown that the bed observed at low flows is not the same as that exists at high flows.

c. Braided channels: Scour in braided channels can reach a maximum at intermediate discharges where the flow in the channel braids attacks the banks at sharp angles.

7.5.2 Toe Protection Methods of Revetment

Toe protection of revetments may be provided by following methods:

a. Extension to maximum scour depth: Lower extremity of revetment placed below expected scour depth or founded on non-erodible bed materials. These are preffered methods, but they can be difficult and expensive when underwater excavation is required.

b. Placing launchable stone: Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack. As the attack and the resulting erosion occur below the stone, the stone is undermined and rolls/slides down the slope, stopping the erosion. This method has been widely used on sand bed streams. Launch slope is less predictable if cohesive material is present, since cohesive material may fail in large blocks. Succesful applications include:

- Window Revetments: riprap placed at top of bank.
- Trench-fill Revetments: riprap placed at low water level.
- Weighted riprap toes: riprap placed at intersection of channel bottom and side slope

7.5.3 Dimension of Launching Apron

Among the various methods, launching apron has been considered to be most economical and common method of toe protection of revetment. Launching apron has been considered to be laid horizontally on the river bed/flood plain at the foot of revetment, so that when scour occurs, the materials will settle and replenish the scour hole on a natural slope. Adequate quantity of stone for the apron has to be provided for ensuring complete protection of the whole of the scoured face. This quantity will obviously depend on the apron thickness, depth of scour and slope of the launching apron. This has been considered below :

Thickness of Launching Apron

Spring (1903) recommended a minimum thickness of apron equal to 1.25 times the thickness of stone riprap of the slope revetment. He recommended further that the thickness of apron at the junction of apron and slope should be the same as that laid on the slope but should be increased in the shape of a wedge towards the river bed, where intensity of current attack is severe and hence probability of loss of stone is greater.

Since the apron stone shall have to be laid mostly under water and cannot be hand placed, thickness of the apron at junction according to Rao (1946) should be 1.50 times the thickness of riprap in slope. The thickness of river end of apron in such case shall be 2.25 times the thickness of riprap in slope.

The face slope of the launching apron may be taken as 2:1 for loose stone as suggested by Spring (1903) and Gales (1938). The dimension of launching apron proposed by different authors may be seen in fig. 7.3.

Size of Stone

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

Length of Launching Apron

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

Quantity of Stone in Apron

Knowing the thickness of apron, the depth of maximum probable scour and the slope of the launched apron, the quantity of apron stone can be assessed. For dimensioning and estimating quantity of stone in apron, Fig. 7.4 may be used.

Recommended Shape

It is recommended to use the shape of launching apron as suggested by T.S.N. Rao (fig.7.4) considering the condition of stone provided at the junction of the apron and shape which appears to be more logical.

7.6 Design of Terminations and Transitions

The experience shows that damage to bank protection often starts at terminations, transitions and joints. Though the problems are well known, there are no more detailed principles or even design rules in the form of formulae to estimate for example the required size of rip-rap, concrete blocks or other cover layer materials. As a general principle the revetment in transition zones or at joint should be of strength equal or greater than the adjoining systems. This can be achieved in one of the following ways :

- Increase the thickness of the cover layer at joints and transitions,
- Grout rip-rap or concrete block cover layers with bitumen and
- Use of concrete edge-strips or boards to prevent damage progression along the structure.

The flank protection between the protected and unprotected areas needs mostly a thicker or grouted cover layer or a concrete edge-strip with some flexible transition.

7.7 Design of Filter

7.7.1 General

The stability of the whole revetment depends strongly on the type and composition of the filter layer. A filter should prevent excessive migration of soil particles, while allowing relatively unimpeded flow of liquid from the soil. The function of a filter is;

- to prevent migration of subsoil particles out of the bank slope (Retention Criteria) and
- to allow at the same time movement of water through the filter (Permeability Criteria).

Moreover, the filter layer has to seperate 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 internal flow in the subsoil caused by wave and current action.

The filter may consists of one of the following types of material:

- granular filter, made of loose, bounded or packed grains, and
- fibre filter, made of synthetic or natural materials.

The design of the filter layer must also take realistic account of construction constraints and consolidation of the subsoil following the construction.

A complete understanding of the properties of the soil adjacent to the filter is essential to proper filter design. The soil may range from clay to gravel. A summary of important soil properties for filter design are:

- soil particle size distribution
- Atterberg limits
- dispersion potential by double Hydrometer
- hydraulic conductivity

Important soil parameters to be calculated using results of the above tests are:

- coefficient of uniformity, C_u, from particle size distribution :

$$C_u = d_{60}/d_{10}$$

- linear coefficient of uniformity, C'_u, from particle size distribution :

$$C'_u = d'_{50}/d'_0 = d'_{60}/d'_{10} = d'_{70}/d'_{20} = d'_{100}/d'_{50} = (d'_{100}/d'_0)^{1/2}$$

- cofficient of curvature, C_c, from particle size distribution :

$$C_c = \frac{d_{30}/d_{10}}{d_{60}/d_{30}} = \frac{(d_{30})^2}{d_{60} \times d_{10}}$$

- hydraulic conductivity of soil, can be measured from directly permeability testing; if testing is not possible, then the hydraulic conductivity can be estimated from the particle size distribution. (fig.7.5)

7.7.2 Granular Filter

Conventional gravity filter design requires consideration of both the retention capability and the permeability of the granular filter.

A complete understanding of the properties of the drainage medium adjacent to the filter and the filter medium is essential to proper filter design. The following is a summary of important drainage media and filter medium properties:

- particle size distribution
- volumetric porosity
- hydraulic conductivity
- angularity
- thickness

The inverted filte shall be designed using the following criteria, (Ref : DRAINAGE PRINCIPLES AND APPLICATIONS - by International Institute of Land Reclamation and Improvement Page -174 & 175).

(a) The gradation of filter should conform to the following rule,

$$\frac{d_{15} \text{ filter material}}{d_{85} \text{ base material}} \leq 5$$

$$\frac{d_{50} \text{ filter material}}{d_{50} \text{ base material}} < p$$

$$\frac{d_{15} \text{ filter material}}{d_{15} \text{ base material}} \leq q$$

Filter type	p	[illegible]
For homogenous and round grains	5-10	6-10
For homogenous sharp grains (Sylhet sand)	10-30	6-20
For graded grains (sized khoa, stone chips)	12-60	3-40

(b) The sieve curves of all layers should be almost parelle[l] in the area of the smaller fractions.

(c) Minimum layer thickness

Sand - 0.10 m
Gravel - 0.20 m
Stone - 2 times stone diameter

An analysis of the retention criteria shows that if the pore spaces in granular filters are small enough to retain the coarsest 15 (i.e. d_{15} size) of the adjacent soil, then the majority of the finer soil particles will also be retained.

It has also been shown that the d_{15} size of both the granular filte and the soil are related to their respective pore sizes. As such the ratio of these sizes reflects the ease with which water wil pass from the soil through the filter.

7.7.3 Fibre Filter

A number of geotextile filter criteria have been proposed by various authors. Most agree that both the retention and permeability criteria should be considered for selecting geotextile filter. Several additional considerations are required for proper design of geotextile filters. The criteria for geotextile filter selection are summarized as follows:

- retention criteria to ensure that the geotextile openings are small enough to prevent excessive migration of soil particles.
- permeability criterion to ensure that the geotextile is permeable enough to allow liquids to pass through it without significant flow impedence.
- anti-clogging criterion to ensure that the geotextile has enough openings so that if soil becomes entrapped within the geotextile and clogs a few openings, the permeability of filter will not be significantly hampered.
- survivability criterion to ensure that the geotextile survives its installation.
- durability criterion to ensure that the geotextile is durable enough to withstand adverse chemicals, ultraviolet exposure, and abrasive environments for the design life.

A complete understanding of the properties of the geosynthetic drainage media and geotextile filter media is essential to proper filter design. Summary of important properties are:

. Geosynthetic drainage media

. volumetric porosity
. hydraulic transmissivity
. thickness

. Geotextile filter media

. apparent opening size
. permittivity
. thickness

PIANC (1987a) provides following general criteria for design of fibre filter :

(a) Retention

. for soils with a uniformity coefficient less than 5:
$0.05 \text{ mm} < O_{90} < 0.7\, d_{90}$

. for soils with a uniformity coefficient grater than 5:
$0.05 \text{ mm} < O_{90} < d_{90}$

(b) Permeability

$$\Psi > 5x10^{9} * k_s * i$$

Where,

Ψ	=	permittivity	(1/s)
k_s	=	permeability of the subsoil	(m/s)
i	=	hydraulic gradient in the soil	

7.8 Placement of Riprap

Common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket. Hand placement produces the most stable riprap revetment because the long axis of the riprap particles are oriented perpendicular to the bank. Steeper side slopes can be used with hand placed riprap than with other placing methods. This reduces the required volume of hard material.

In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. stone should not be dropped from an excessive height or dumped and spread, as this may cause segregation and breakage of stone.

The layer thickness and stone size may be increased somewhat to offset the short comings of placement method. Thickness of underwater placement should be increased by 50% to provide for the uncertainties associated with the type of placement.
Underwater placement is usually specified in terms of weight of stone per unit area, to be distributed uniformly and controlled by a "grid" established by shoreline survey.

Table - 7.1

Suggested Stone Rip-Rap Grading for Stream Bank Revetment

Class - I

Nominal 12 inch diameter or 80 lb weight
Allowable local velocity up to 10 ft/sec

Grading Specification

100% smaller than	18	inches or	300 lb
At least 20% larger than	14	inches or	150 lb
At least 50% larger than	12	inches or	80 lb
At least 80% larger than	8	inches or	25 lb

Class - II

Nominal 20 inch diameter or 400 lb weight
Allowable local velocity up to 13 ft/sec

Grading Specification

100% smaller than	30	inches or	1500 lb
At least 20% larger than	24	inches or	700 lb
At least 50% larger than	20	inches or	400 lb
At least 80% larger than	12	inches or	70 lb

Class - III

Nominal 30 inch diameter or 1500 lb weight
Allowable local velocity up to 15 ft/sec

Grading Specification

100% smaller than	48	inches or	5000 lb
At least 20% larger than	36	inches or	2500 lb
At least 50% larger than	30	inches or	1500 lb
At least 80% larger than	20	inches or	400 lb

Table - 7.2

Spring's Table to Compute Thickness of Stone on Slope

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

Table - 7.3

Gales' Table to Compute Thickness of Stone on Slope

River	Rivers with discharge 0.25 to 0.75 million cusec		Rivers with discharge 0.75 to 1.5 million cusec		Rivers with discharge 1.5 to 2.5 million cusec	
Parts of guide bank	Head	Body & Tail	Head	Body & Tail	Head	Body & Tail
Pitching Stone	3'- 6"	3'- 6"	3'- 6"	3'- 6"	3'- 6"	3'- 6"
Thickness of Soling Ballast	7"	7"	8"	8"	9"	9"
Total Thickness	4'- 1"	4'- 1"	4'- 2"	4'- 2"	4'- 3"	4'- 3"

CURVES TO DETERMINE THE SIZE OF STONES NEEDED FOR SLOPE PROTECTION

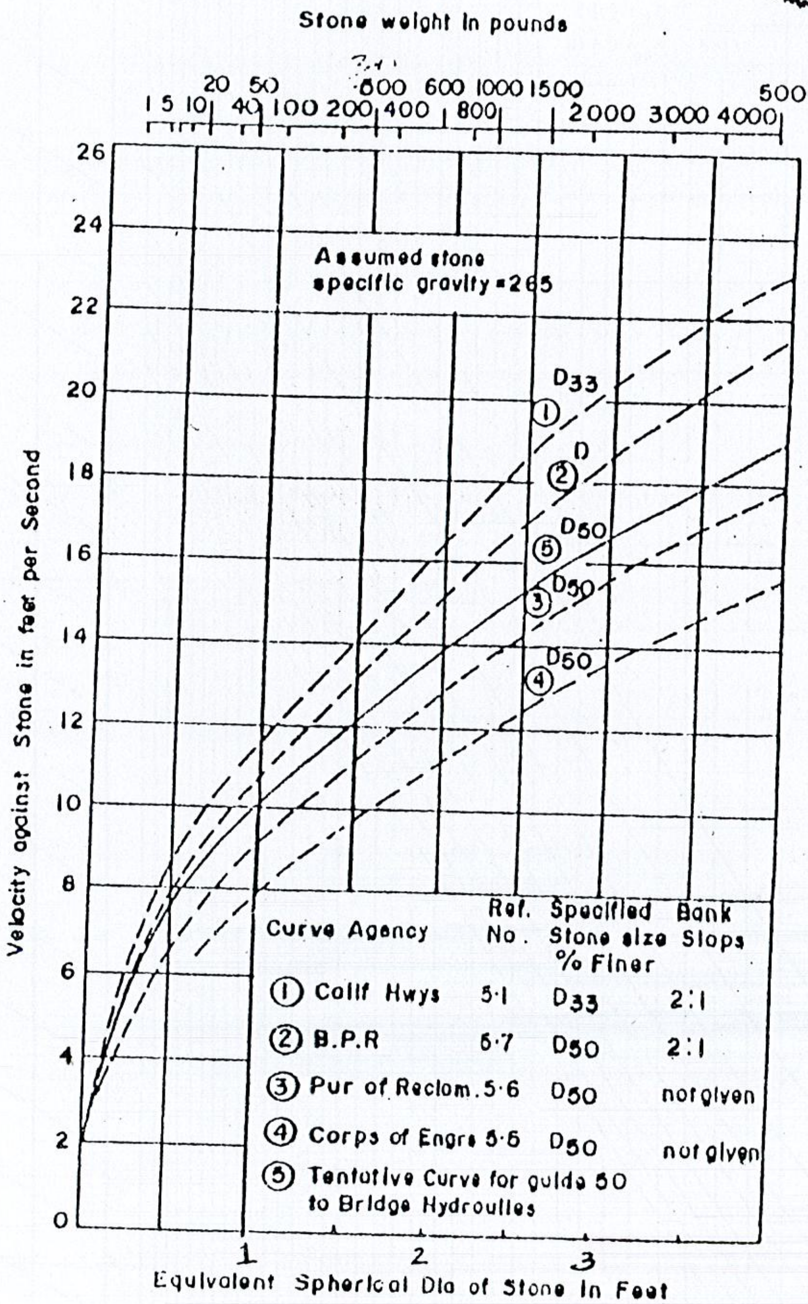
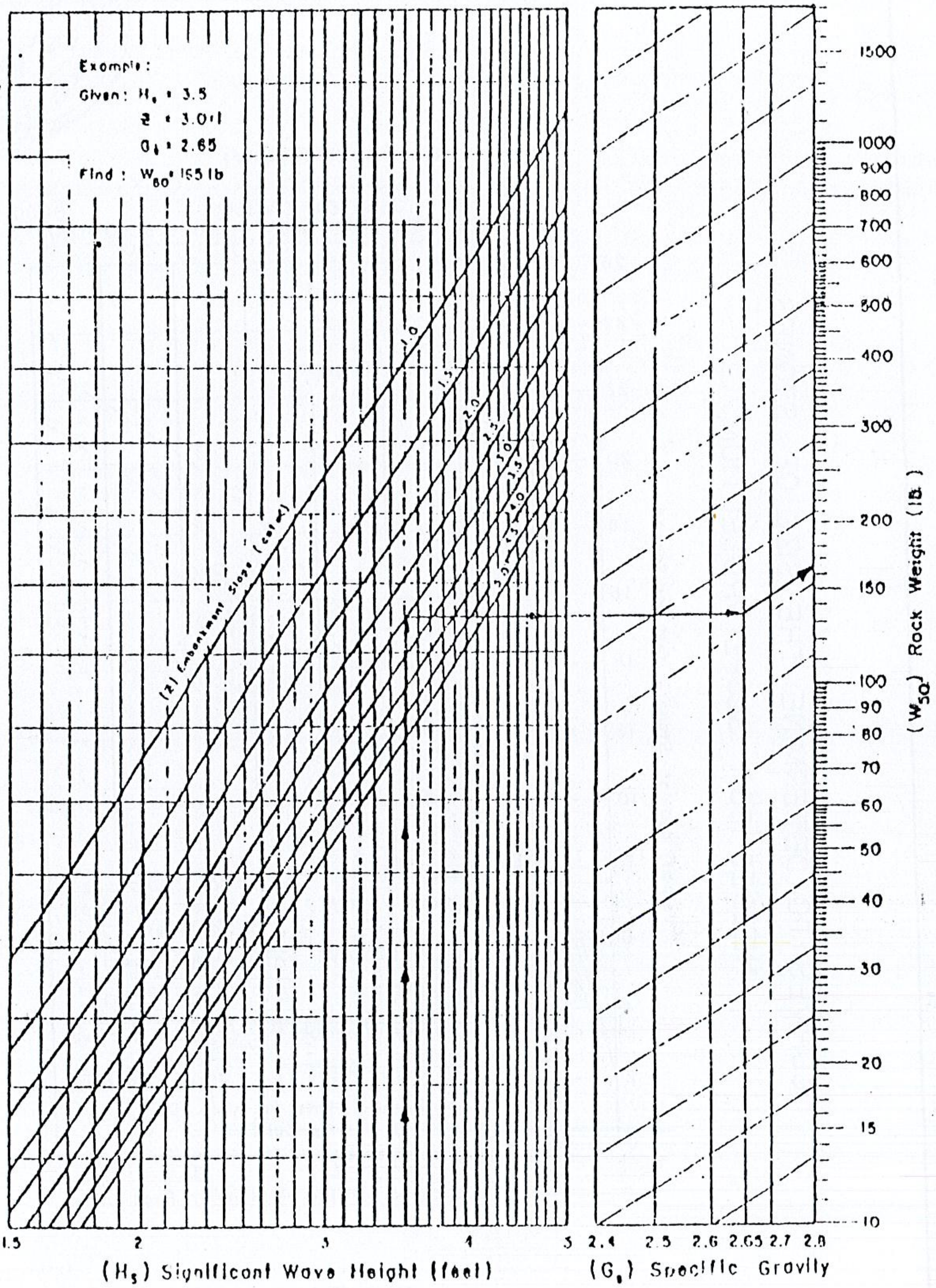


FIG. 7·1



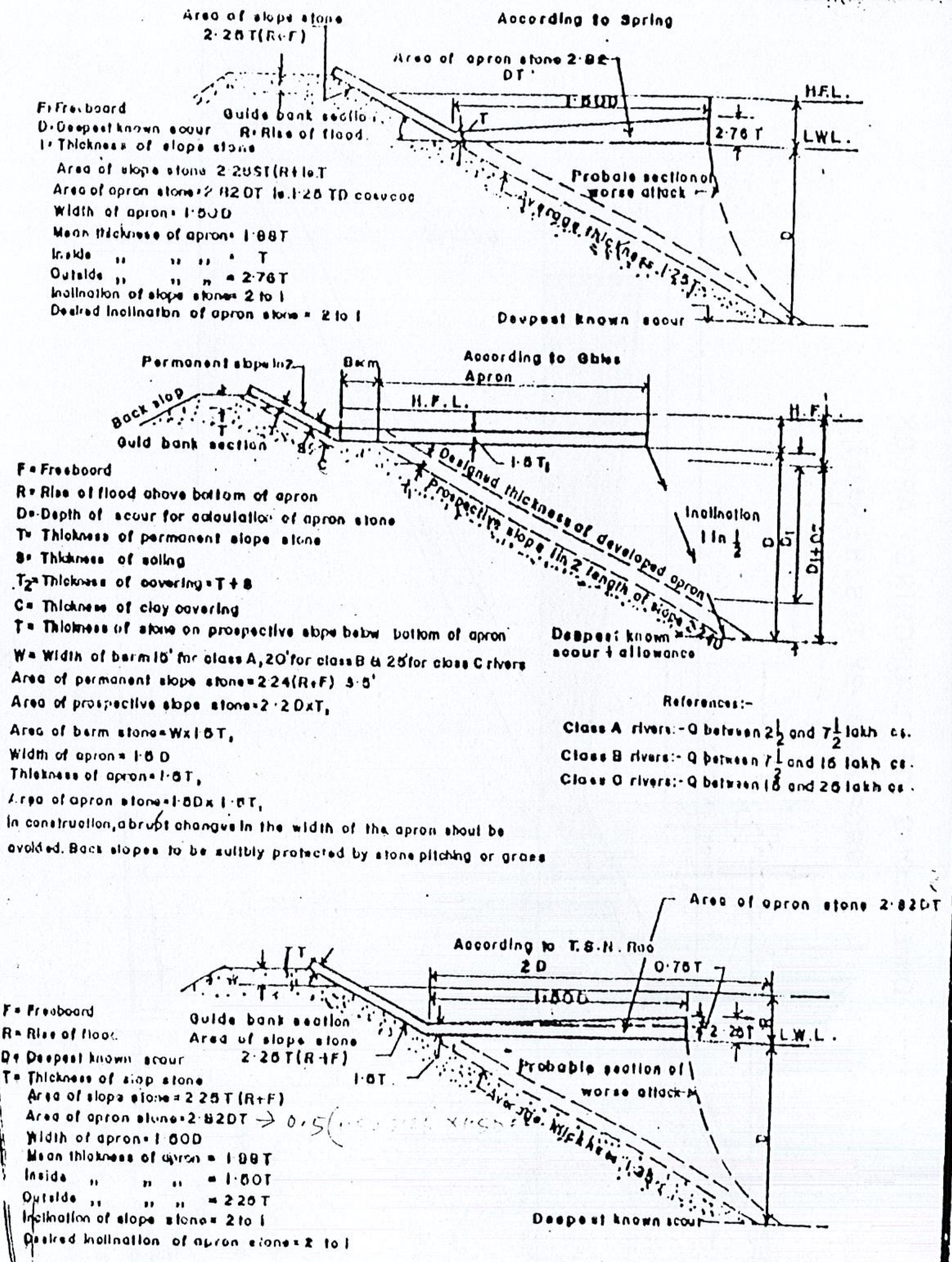
Equation (E 14); From Appendix E

$$W_{50} = \frac{19.5\ G_s\ H_s^3}{(G_s - 1)^3\ \cot\alpha}$$

FIG. 7·2

ROCK
SIZE SELECTION

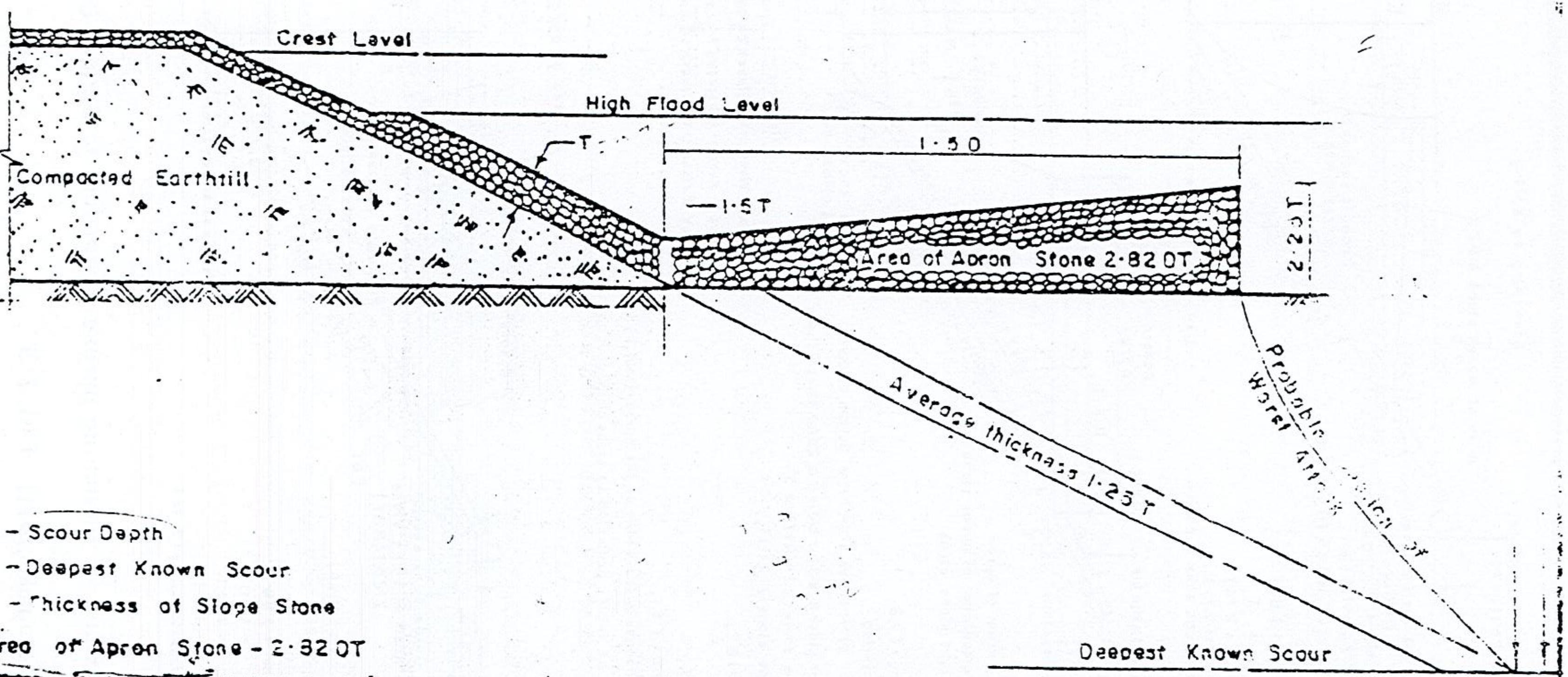
WHTC ENG. STAFF



Dimensions of launching aprons according to Spring, Gales and Rao
(Source: Joglekar, 1971) FIG. 7·3

DIMENSIONS OF LAUNCHING APRON

(PROPOSED BY T. S. N. RAO)



R – Scour Depth

D – Deepest Known Scour

T – Thickness of Slope Stone

Area of Apron Stone – 2·82 DT

Width of Apron – 1·50D

Mean Thickness of Apron – [illegible]

Inside Thickness of Apron – 1·50T

Outside Thickness of Apron – 2·25 T

De[illegible]ed Inclination of Apron Stone 2 to 1

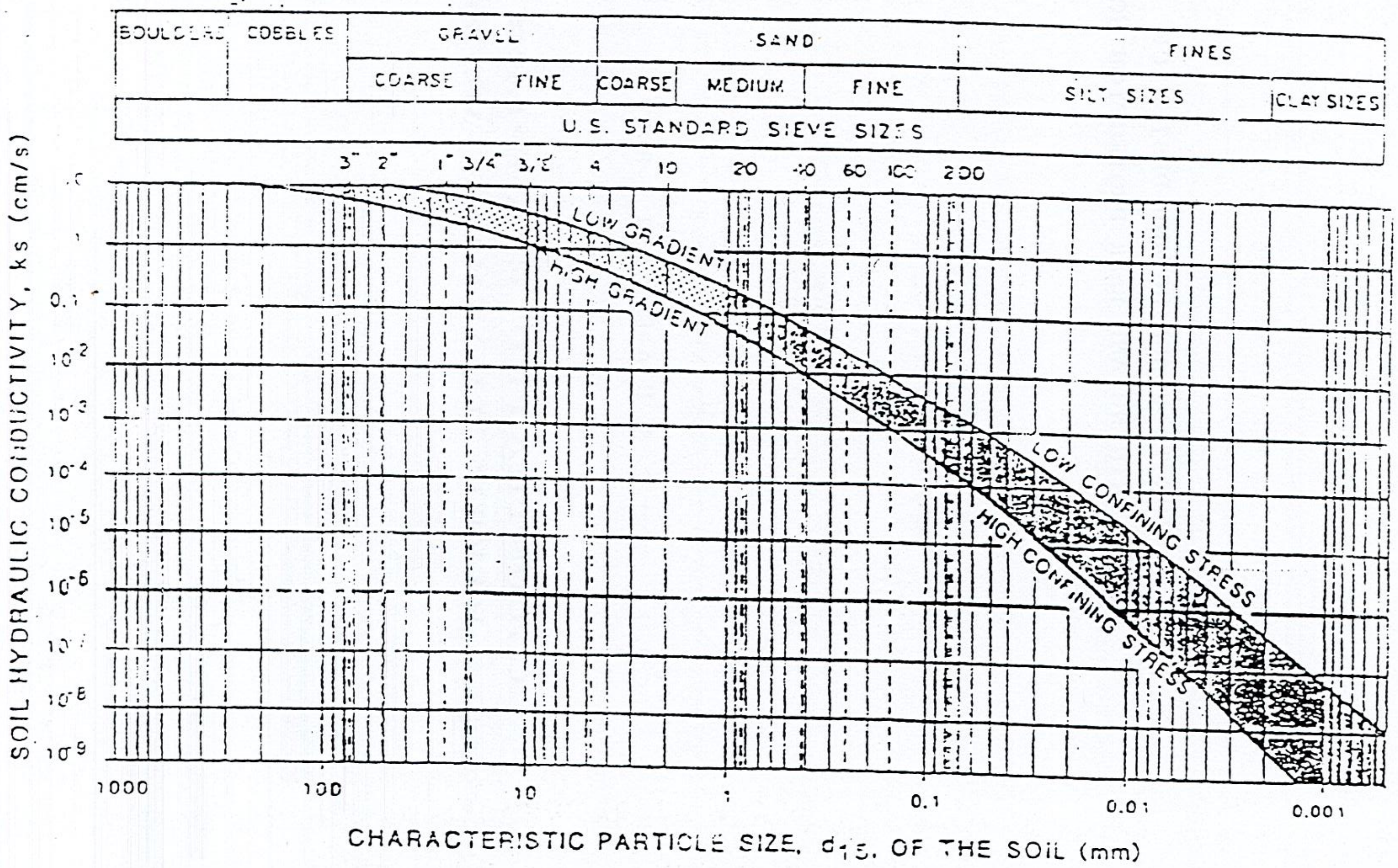
FIG – 7·4

FIG – 7·4

FIG-7.4

FIGURE 7.5

TYPICAL HYDRAULIC CONDUCTIVITY VALUES



GUIDE TO PLANNING AND DESIGN OF
RIVER TRAINING AND BANK PROTECTION WORKS

CHAPTER 8

ILLUSTRATED EXAMPLES ON RIVER TRAINING WORKS

CHAPTER 8

8.0 Illustrated Examples on River Training Works

Design for Bank Revetment

8.1 Design Data :

The following data have been prepared for the Bhairab Bazar site on Meghna River :

• Discharge - Bankfull		:	16,000 cumec
100 year flood		:	18,000 cumec
• High Water Level - Bankfull		:	7.00 m PWD
100 year flood		:	7.89 m PWD
• Average Low Water Level		:	1.22 m PWD
• Average Flow Velocity		:	3.00 m/sec
• Water Surface Slope		:	4.74 cm/km (3"/mile)
• Average size of bed material		:	0.052 mm
• Average River Bed Level		:	(-) 12.50 m PWD
• Revetment material		:	Boulder
• Specific Gravity, S_s		:	2.65
• Wind Speed		:	15 m/sec
• Wind Duration		:	1.00 hour
• Fetch Length		:	5.0 km
• Ratio of water depth and Revetment size,	h/D	:	5
• Shields Constant,	ψ	:	0.0275
• Slope of Bank,	θ	:	26.5°
• Angle of Repose of Revetment Material,	Φ	:	40°

8.2 Design Computation

(A) Design for size of Revetment Material ([illegible])

(i) Against Velocity

(a) Using Neill's method (Refer. Page - 114)

$$D = 0.034\ V^2$$

$$= 0.034 \times 3.00^2$$

$$= 0.306\ \text{m}$$

$$= 306\ \text{mm}$$

(b) Using JMBA Equation (Refer. Page - 115)

$$Dn = \frac{0.7\ V^2}{2(S_g - 1)g} \times \frac{2}{[\log\ (6h/D)]^2} \times \frac{1}{[1-(\sin\theta/\sin\Phi)^2]^{0.5}}$$

$$= \frac{0.7 \times 3^2}{2(2.65-1)9.81} \times \frac{2}{[\log(6x5)]^2} \times \frac{1}{[1-(\sin 26.5^\circ/\sin 40^\circ)^2]^{0.5}}$$

$$= 248\ \text{mm}$$

$$D = \frac{Dn}{0.81} = \frac{248}{0.81}$$

$$= 306\ \text{mm}$$

P= 67

(ii) Against Wave

From Table 4.1, For wind speed of 15 m/sec with 1.00 hour duration and 5.0 km Fetch

$H_s = 0.7$ m

$T = 2.8$ sec

$$E = 1.25 \frac{T}{(H_s)^{1/2}} \tan \theta \quad \text{(Refer. Page - 120)}$$

$$= 1.25 \frac{2.8}{(0.7)^{1/2}} \tan 26.5^\circ$$

$$= 2.086$$

(a) Using Pilarczyk equation, (Refer. Page - 78)

$$\text{Dimension of cube} = D = \frac{H_s}{S_s - 1} * \frac{1}{\beta} * \frac{E^{1/2}}{\cos\theta}$$

$$D = \frac{0.7}{2.65 - 1} * \frac{1}{3} * \frac{\sqrt{2.086}}{\cos 26.5^\circ}$$

$$= 228 \text{ mm.}$$

Equivalent Stone Diameter = 228/0.81 = 282 mm

Comparing the size of stone required against velocity and wave 300 mm dia stone is recommended.

(B) Design for Thickness of Rip-Rap

(i) Using Inglish Formula (Refer. Page - 129)

$T = 0.06\ Q^{1/3}$

$= 0.06\ (16,000 \times 35.32)^{1/3}$

$= 4.96$ ft

$= 1488$ mm

P. 143

(b) According to Spring, for a river slope of 3" per mile (4.74 cm per km) with fine and very fine bed materials are as follows :

Fine bed materials : 34" = 850 mm

Very fine bed materials : 40" = 1000 mm

(c) According to Gales, for a river discharge with 0.25 million to 0.75 million cusec :

Stone Pitching Thickness : 3'- 6" = 1050 mm

(d) Based on stone size :

According to ESCAP, Thickness of Protection

$= 1.5\ D = 1.5 \times 300 = 450$ mm

Inglish Formula always provides higher thickness of protection, while it is lower based on stone size. Considering Meghna as large river, the following thickness of Rip-Rap is recommended based on Spring and Gales values,

Above Low Water Protection : 1000 mm

Under Water Protection : 1500 mm (50% higher in case of dumping)

(C) Design of Launching Apron

- The apron is assumed to launch in 1:2 slope
- Length of Apron equal to 1.5 times Scour Depth below bed level
- Scour Multiplying Factor = 1.5 (considering minor river bend)

Silt Factor $f = 1.76\,(d_s)^{1/2}$ [d_s in mm]

$= 1.76\,(0.052)^{1/2}$

$= 0.40$

Scour Depth, $R = 0.47\,(Q/f)^{1/3}$

$= 0.47\,(16000/0.4)^{1/3}$

$= 16.07$ m

Design Scour Depth = 1.5 R = 1.5 x 16.07

= 24.11 m

Scour Level = (Design Water level) - (Design Scour Depth)

= 7.00 - 24.11

= (-) 17.11 m (PWD)

Depth of Scour,

D = (Average river bed level) - (Scour level)

= (-) 12.5 - (-) 17.11 = 4.61 m

Length of Launching Apron,

L = 1.5 D

= 1.5 x 4.61

= 6.92 m

Thickness of Launching Apron as per T.S.N. Rao,

- At toe end = 1.50 x 1500 = 2250 mm
- At River end = 2.25 x 1500 = 3375 mm

(D) Design for Grading of Rip-Rap Materials

Based on Table 7.1, Gradation of Stone are as follows :

100% smaller than	400 mm
At least 20% larger than	350 mm
At least 50% larger than	300 mm
At least 80% larger than	250 mm

OR

200 mm to 250 mm	20%
250 mm to 300 mm	30%
300 mm to 350 mm	30%
350 mm to 400 mm	20%

The following Gradation is recommended :

250 mm to 300 mm	30%
300 mm to 350 mm	50%
350 mm to 400 mm	20%

(E) Design of Filter

(a) Base Material :

d_{15} : 0.09 mm
d_{50} : 0.19 mm
d_{85} : 0.23 mm

(b) Design for Filter Material - 1

Criteria -1a, $\frac{d_{15} \text{ Filter Material}}{d_{85} \text{ Base Material}} <= 5$

Hence, d_{15} Filter Material = 0.23 x 5 = 1.15 mm

Criteria -1b, $$\frac{d_{15} \text{ Filter Material}}{d_{15} \text{ Base Material}} < (6\text{-}20)$$

Hence, range of d_{15} Filter Material = 0.09 x 6 to 0.09 x 20
= 0.54 mm to 1.80 mm

Criteria -2, $$\frac{d_{50} \text{ Filter Material}}{d_{50} \text{ Base Material}} < (10\text{-}30)$$

Hence, range of d_{50} Filter Material = 0.19 x 10 to 0.19 x 30
= 1.90 mm to 5.70 mm

Hence from Fig. 8.1 Filter Material - 1 :

d_{15} : 1.75 mm
d_{50} : 2.50 mm
d_{85} : 4.00 mm

(c) Design for Filter Material - 2

Criteria -1a, $$\frac{d_{15} \text{ Filter Material}}{d_{85} \text{ Base Material}} <= 5$$

Hence, d_{15} Filter Material = 4.00 x 5 = 20.00 mm

Criteria -1b, $$\frac{d_{15} \text{ Filter Material}}{d_{15} \text{ Base Material}} < (12\text{-}40)$$

Hence, range of d_{15} Filter Material = 1.75 x 12 to 1.75 x 40
= 21.00 mm to 70.00 mm

Criteria -2, $$\frac{d_{50} \text{ Filter Material}}{d_{50} \text{ Base Material}} < (12\text{-}60)$$

1

Hence, range of d_{50} Filter Material = 2.50 x 12 to 2.50 x 60
= 30.00 mm to 150.00 mm

Hence from Fig. 8.1 Filter Material - 2 :

d_{15}	:	17.02 mm
d_{50}	:	36.43 mm
d_{85}	:	60.21 mm

U/S end termination

xR = 2.75

Y + D = xR

V = (3.25 + 21.095) x 21.26 x   D = XR − Y

HFL

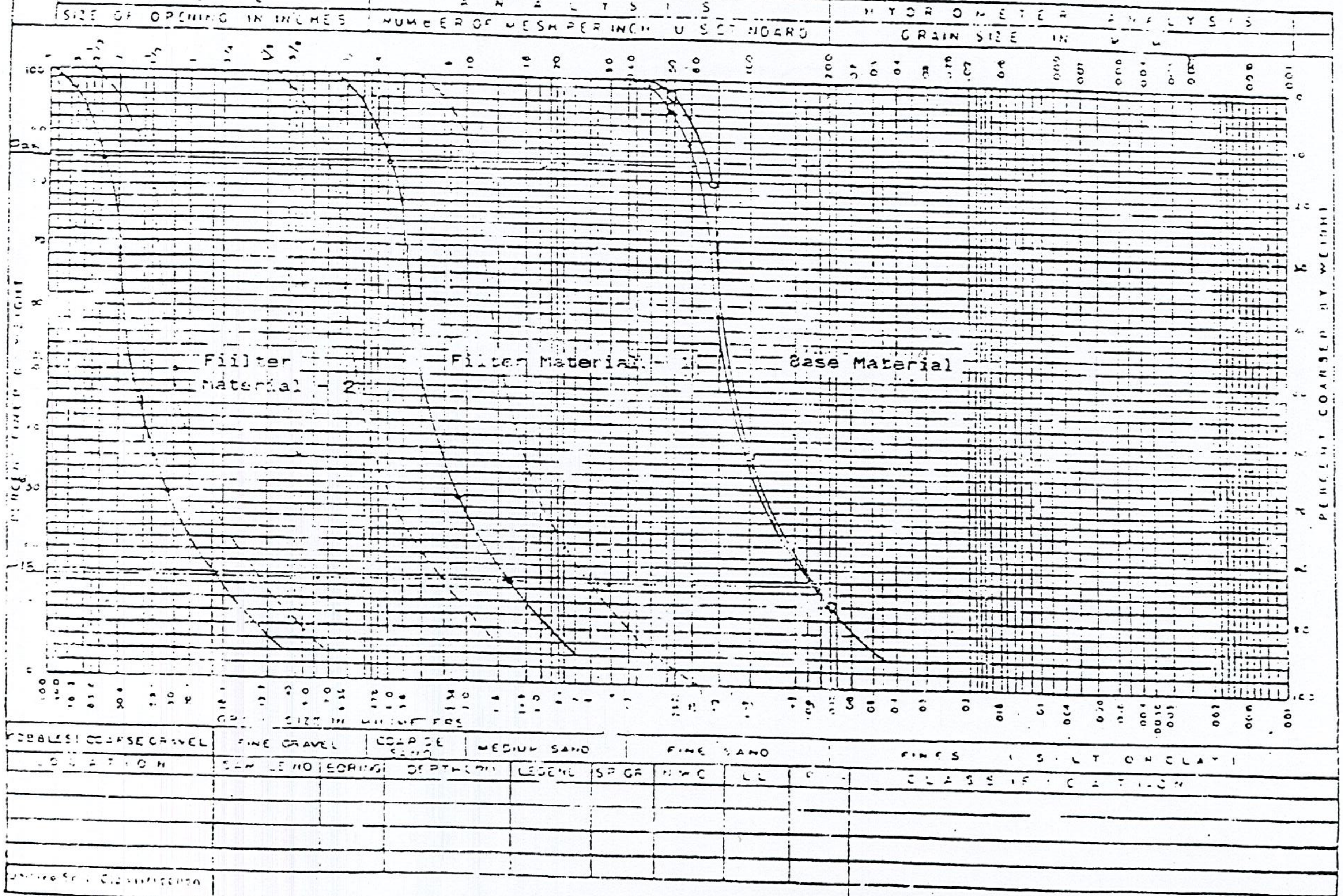
x = 2 for U/S termination

x = 1.75 for d/s termination

x = 1.5 for shank

GRAIN SIZE ANALYSIS CURVE

FIG. 8·1



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