

FLOOD PLAN
COORDINATION
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KREDITANSTALT FÜR
WIEDERAUFBAU (KfW)

CAISSE FRANCAISE
DE DEVELOPPEMENT (CFD)

Call - 796
FAP-21/22

**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

16

**DRAFT FINAL REPORT
PLANNING STUDY**

BW-649
D-796

VOLUME VII



- ANNEX 15: Special Scour Investigations**
- ANNEX 16: Design of Cover Layers of Revetment**
- ANNEX 17: Design of Filter Layers of Revetment**
- ANNEX 18: Dimensioning of Groynes**
- ANNEX 19: Constructional Aspects**
- ANNEX 20: Study of Alternative Design Solutions**
- ANNEX 21: Preliminary Design of Test Structures**
- ANNEX 22: Financial Analysis**

JANUARY 1993



CONSULTING CONSORTIUM FAP 21/22

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Volume IV*	Annex 7 Socio-Economic and Administrative Assessment (Part A, B) Annex 9 Economic Assessment Annex 10 Navigation Study
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*) Submitted with the FAP 22 Draft Final Report

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**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

DRAFT FINAL REPORT

ANNEX 15

SPECIAL SCOUR INVESTIGATIONS

JANUARY 1993

SPECIAL SCOUR INVESTIGATIONS

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LIST OF SYMBOLS

A	= Amplitude of meander; Coefficient which weighs the influence of spiral flow
B	= Channel width
B_s	= Width of a scour hole
b	= Width of spur dike
b	= Exponent in sediment transport formula
C	= Chézy coefficient
C_D	= Drag coefficient
c	= A subscript related to the channel centre
D_{50}	= Mean grain size
D_{cr}	= Diameter of the particle whose motion is impending
d_{sc}	= Scour depth (in equilibrium)
E	= Calibration coefficient; An erodibility coefficient for bank erosion due to excess bank shear stress
E_h	= Erodibility coefficient for bank erosion due to excess near bank depth
E_u	= Erodibility coefficient for bank erosion due to excess near bank flow velocity
F	= Friction term
F_D	= The particle densimetric Froude number = $U/\sqrt{\Delta g D}$
h_a	= Average depth of upstream anabranches
F_r	= Froude number based on u, g and h
f	= Silt factor according to Lacey
h_{bf}	= Bankfull depth
f	= Darcy-Weisbach friction factor
h_i	= Initial depth before local scour
h_o	= Depth of uniform flow
f_s	= A slope factor of the grains, usually with a value between 1 and 2
$f(\theta)$	= A function which weighs the influence of transverse bed slope.
h_r	= Regime depth according to Lacey
g	= Acceleration due to gravity
h_{sb}	= Water depth near outer bank of a bend
h_{sl}	= Water depth at the head of a groyne on the scour hole bottom
H	= Total bank height
I	= A measure of the intensity of spiral flow
i	= Energy slope
i_T	= Transverse bed slope at centre line
H_o	= Amplitude of oscillation at the beginning of bend
h	= Water depth
K	= Factor for flow and geometric conditions
L	= Meander arc length
L_D	= Length scale of damping

L_p	= Wave number of oscillation
m	= Friction parameter based on the ratio between average flow velocity, von Karman constant and shear velocity (U_*)
n	= The component, perpendicular to flow direction; Velocity profile exponent
n_x	= Scale of a parameter (x)
o	= A subscript denotes fully developed values
p	= Pressure
Q	= Discharge
q	= Discharge per unit width of channel
R	= Radius of curvature
R_s	= Radius of streamline curvature
R_n	= Radius of normal line curvature
R_*	= The effective local radius of curvature of the streamline
R_{*cr}	= Critical boundary Reynolds number
r	= Distorsion of model
r	= Sinuosity = $2L/\lambda$
S	= Bed load transport per unit width
s	= The component in flow direction
t	= Time
U	= Flow velocity
$\bar{v}$	= Average main velocity
U_*	= Shear velocity
U'	= Disturbance at the upstream end of bend (m/s)
u, v, w	= Velocity components in s, n and z directions, respectively
$\bar{U}_{cr}$	= Critical mean flow velocity
w	= Fall velocity of a grain of bed material
X	= Transport parameter
x	= Rectangular Cartesian coordinate
Y	= Flow parameter
y	= Rectangular Cartesian coordinate
z_b	= Bed level above a reference level
z_w	= Water level above a reference level
α	= Coefficient; ratio of projected surface area to volume for a non-spherical particle divided by that for a sphere (≈ 1.27); Opening ratio = $(B-b)/b$
α_1	= Spur dike angle
β	= Transverse or radial slope of bed surface
β	= Coefficient
Δ	= Relative density of sediment = $(\rho - \rho_s)/\rho$
Δn	= Rate of lateral erosion
Δv	= Volume of bank erosion products per unit length of channel
Δz_w	= Difference in water level between bankfull and flood level

δ	= Direction of bed shear stress
Δh_b	= Maximum depth of scour in outer bend
Δh_1	= Maximum depth of scour at the head of groyne in a straight channel
Δh_{1b}	= Maximum depth of scour at the head of groyne in a curved channel
ϵ	= Calibration coefficient
η_o	= Relative effective bank height
θ	= Shields parameter
θ	= Angle between confluencing anabranches
$\bar{\theta}$	= Mean Shields parameter
θ_{cr}	= Critical shields parameter
κ	= von Karman constant
λ	= Wave length of the meander
λ_s	= Adaptation length of bed deformation
λ_w	= Adaptation length of main flow
μ	= Ripple factor = $(C/C_{\text{grain}})^{1.5}$
ρ	= Density of fluid
ρ_s	= Density of sediment
τ	= Shear stress
τ_{cr}	= Critical shear stress of the bed material
φ	= Bank slope
ψ	= Arc angle
ω	= Wash load factor; Vorticity



1 GENERAL

1.1 INTRODUCTION

This Annex provides a description of the scour investigations carried out to arrive at a better predictions for the scour near groynes in outer bends. This study is tackling one of the questions in relation to the prediction of combined scour. In this Annex the relevant literature is summarized and a description is given of the test facilities used, the results obtained and the recommendation arrived at, regarding the prediction of scour near groynes in outer bends.

1.2 COMBINED SCOUR

Until recently maximum scour depths near river training works in Jamuna type of rivers were usually determined using methods essentially based on the regime approach as proposed by Ahmad (1953) and others. In these methods only limited use is made of the specific understanding of river processes. In fact when the design methods proposed by Ahmad and others were developed, the understanding of phenomena in particular braided rivers like the Jamuna river was only limited.

Since then there has been quite some progress in the understanding of river processes. One dimensional mathematical models have been developed to simulate **general scour**, as the reaction of the river to changes in its boundary conditions (like deforestation, diversions, sea level rise, etc.) as defined in Jamuna Bridge Project Study (1990). The understanding of **scour in bends** has substantially improved, not only for axisymmetric conditions but also for the conditions in the beginning of a bend, including the so called overshoot and point bar formation (see Struiksma et al., 1985). Much more insight is obtained into the influence of bank erosion products on the lateral bed profile (see Jamuna Bridge Project Study, 1990 and Mosselman, 1992). Expressions for **constriction scour** have been developed, including their timewise development via numerical models or via analytical solutions (see Klaassen and Struiksma, 1988). **Confluence scour** has been identified (Ashmore and Parker, 1983), and special predictors have been developed for Jamuna type of rivers (Klaassen and Vermeer, 1988a). Insight into bedform dimensions in large rivers like the Jamuna river and the related minimum **bedform scour depth** was obtained and their relevance for the design of river training works was indicated (Klaassen and Vermeer, 1988b). Finally a number of physical model studies have been carried out allowing to derive expressions for **local scour** (Pilarczyk, 1984; Breusers and Raudkivi, 1991).

It is challenging to use this improved understanding to arrive at a better prediction of design scour depths for river training works along Jamuna type of rivers. This is especially

important when a probabilistic design approach is used, because for all contributions to the total scour depth separate probability density functions can be derived, which can or cannot be correlated.

Indeed, recently some projects have been carried out which are trying to use this increased understanding to arrive at more scientifically based predictions of the scour depths that may occur near river training works that are being planned along the Jamuna and similar types of rivers. It is interesting to summarize here the approaches taken within these different projects, especially because it appears that being able to differentiate between the different types of scour and to be able to compute their contribution to the total scour depth is a necessary condition, which is however, not sufficient for establishing the design criteria for river training works.

If the different contributions are estimated separately, then the major issue becomes how to combine them. There is at present no univocal opinion of how to do this and in fact this is the very topic of the present study, to make combination of bend scour and local scour around groynes.

Within the framework of the present study a review was made of the approaches taken in three different projects which were carried out in Bangladesh recently and which can be cited to illustrate that till now not one established method is available to combine these different types of scour. The three projects referred to here are:

- (1) Jamuna Bridge Project Phase I and Phase II Study;
- (2) Brahmaputra River Training Studies (FAP 1, 1991), and
- (3) Meghna River Bank Protection Short Term Study (FAP 9B, 1992).

During the review it was observed that different approaches have been used for the different sites and that there is no univocal opinion on the importance of some of the contributions to the total scour. Also the way some of these various contributions were taken into account was slightly different.

The review has led to the following questions to as how to determine extreme scour depths in Jamuna type of rivers and how to combine the different contributions to the total scour:

- (1) Should evaluation of the bend scour be based on observed bend profiles or should the influence of the erosion of bank erosion products be taken into account ?
- (2) Is local scour in front of revetments negligible (as assumed in FAP 1) or does it have a significant contribution to the total scour (as assumed in FAP 9B) ?
- (3) Is protrusion scour indeed an important phenomena worthwhile to be identified and assessed independently (as done in the FAP 9B) ?
- (4) Is bedform scour important and should it be taken into account (as done in the FAP 1) or is its influence negligible near structures where local scour is dominant (as assumed in the Jamuna Bridge study and FAP 9B) ?

- (5) Should local scour (and protrusion scour) be computed including the effect of the constriction scour and the bend scour on the initial water depth (FAP 9B) or should it be evaluated separately and simply be added to the combined scour excluding local scour (as assumed in Jamuna Bridge study) ?

The present study addresses specifically the fifth question and to a smaller extent the first question, leaving the other questions open for further studies.

It should be mentioned here that the prediction methods employed during the different projects are all based on an extensive analysis of field data, during which the combined scour that had been observed in the past was split up in the different contributions to scour. It is therefore felt that any differences in conception as to how to combine these contributions will have a minor effect on the predictions made for those particular locations. Application of the approach taken and the methods developed to other locations should be done with care, however, until answers can be given to the questions listed above.

1.3 APPROACH

In principle different approaches can be used to study the combined scour around groynes in outer bends, notably a theoretical study, a study in the field and a laboratory investigation in a scale model. Here the last option was selected: a theoretical study seems less appropriate in view of the complex 3-dimensional flow pattern near groynes and in bends, and a field investigation always suffers from the impossibility to be imposed on the boundary conditions.

A special facility was constructed for this study, and in this facility experiments were carried out with straight and curved channels as well as with and without groynes. A major difference with previous studies addressing local scour is that these experiments were done with an actively mobile bed, hence the bed topography in the bend could develop in a natural way. In this respect these experiments are deviating substantially from the previous studies, where no major bed level changes did occur, like Ahmad, 1953.

The set-up of the experiments was such that three different series of experiments were carried out, keeping the hydraulic conditions approximately the same for each series. This allowed to compare the experiments mutually in one series, and to identify the magnitudes of bend scour and local scour individually, and then to determine how they should be combined to arrive at the same values as observed in the experiments with groynes along outer bends.

2 LITERATURE SURVEY ON SCOUR IN BENDS AND SCOUR NEAR GROYNES

2.1 GENERAL

The bed topography in an alluvial river is established by the interaction between the flow and the sediment transport. One of the main problems in bends is the fact that the flow is curved. A characteristic feature of curved flows is the spiral current (or helical flow), which causes a difference between the direction of bed shear stress and the direction of depth averaged flow. Moreover, pronounced bed topography and flow variations cause spatial variation of the grain size of the river bed in case of graded bed material.

River bank erosion is one of the most important and common phenomena of bends causing a considerable amount of eroded products and river migration. Groynes are normally used for protection of river banks, but scour holes around the head of groynes have to be considered for the sake of stability of groynes, in particular for the determination of the depth of the groyne foundation. Otherwise the structure might be damaged and consequently become useless.

In this chapter, a review of the previous works on bend scour, local scour and combined scour is presented. Some methods or predictors are selected for prediction of each type of scour. The results of this survey were used for the design of the experiments and during the analysis of the results.

2.2 BEND SCOUR

2.2.1 Characteristics of Bends

The morphology of a river bend is one of the key problems in river engineering and geomorphology and it has attracted the attention of scientists and researchers for quite some time.

The Consultant prefer to give a geometrical characteristic of bends before describing their hydraulic and geomorphological characteristics. Various measures have been introduced by many authors so as to define the plan geometry of regular meandering channels. The most important characteristics are shown in Fig. 2.2-1.

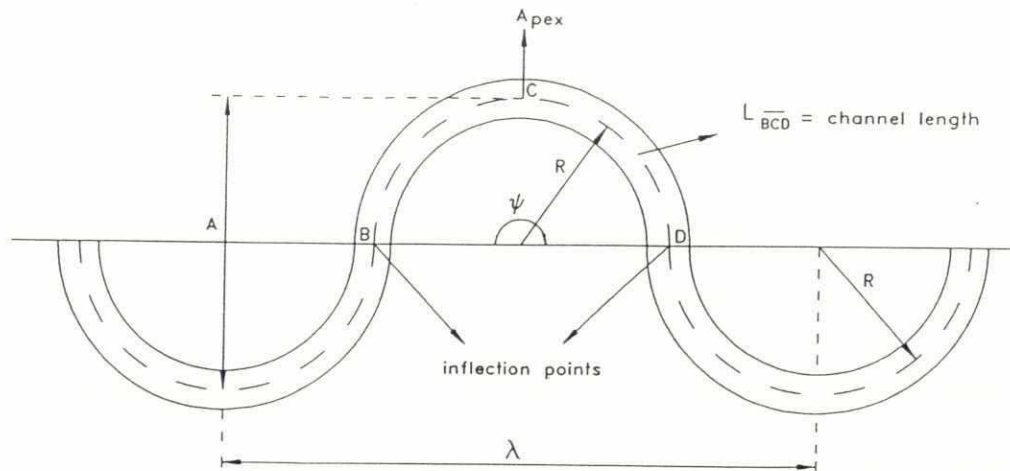


Fig. 2.2-1: Definition sketch of bend geometry

in which

- λ = wavelength, is defined as the distance between the two apices of the meander pattern
- A = amplitude, is the distance between the tangents to the axis at both arms near to or at the point where the sections for obtaining the meander length are marked
- R = radius of curvature, is the radius of the circle defining the curvature of an individual bend measured between adjacent inflection points
- L = meander-arc length, is the distance swept out by the radius of curvature between adjacent inflection points
- ψ = Arc angle, is the angle swept out by the radius of curvature between adjacent inflection points
- r = sinuosity, is the ratio between the channel length and the valley axis length

2.2.2 Most Important Phenomena in Bends

The most important of the hydraulic and geomorphological phenomena in bends can be specified as follows:

a. Water surface slope

In addition to the longitudinal slope, the free water surface in bends develops a transverse slope where the level of the water surface at the concave bank is higher than that at the convex bank (Fig. 2.2-2). This head gradient balances the centrifugal force produced in the bend. It can be expressed by equation (2.2-1) under the following assumptions that static pressure exists in the vertical, as defined by:

$$\frac{dh}{dR} = \alpha \frac{\bar{v}^2}{gR} \quad (2.2-1)$$

in which

R = radius of curvature at the verticla (m)

$\bar{v}$ = average main flow velocity over a vertical (m/s)

α = empirical coefficient (-)

In general the value of α is assumed at 1.06.

b. Transverse secondary current

The most prominent feature of flow in bends is a strong helical motion which originates from the combination of main flow in longitudinal direction and transverse secondary current. The appearance of a secondary current in the transverse direction is attributed to the friction at the bottom of the channel which causes the variation of velocity along the vertical from a minimum near the bed to a maximum near the water surface. This means that equation(2.2-1) holds only at one point along the vertical where the local velocity equals the average one for the vertical, ($v = \bar{v}$).

Near the water surface the local velocity is greater than the average one ($v > \bar{v}$); therefore, the right hand side of equation (2.2-1) becomes larger than the left hand side, meaning that the centrifugal force is greater than the hydrostatic pressure, hence the water particles will be displaced towards the outer direction. Near the bed ($v < \bar{v}$), so this will induce the water particles to move towards the inner bank. These deviations result in a rotation around an axis in the direction of the main flow, which is known as the secondary current (see Figs. 2.2-2 and 2.2-3).

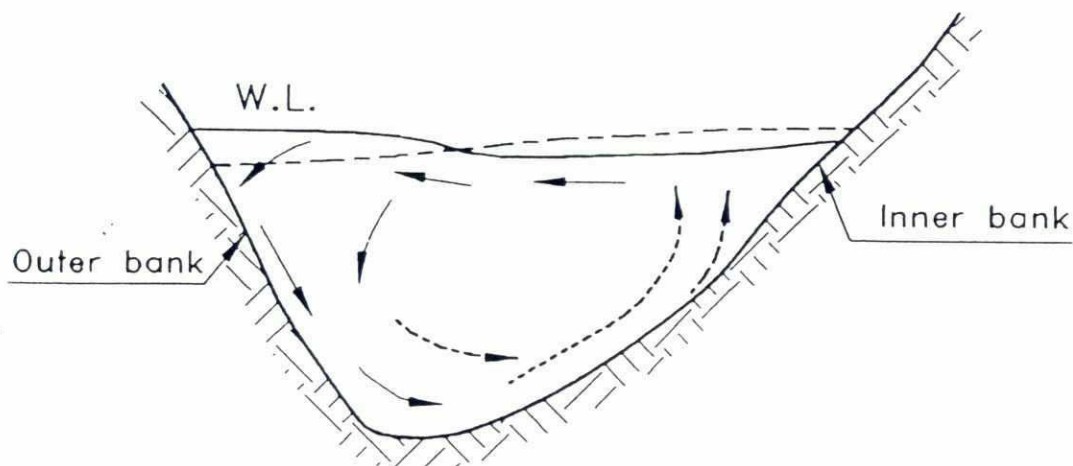


Fig. 2.2-2: Water surface and secondary current in bend

c. Redistribution of velocities

Following the appearance of the secondary current in a stream, there is a considerable change in the velocity distribution both along the vertical and across the width of the channel. The first one is due to the exchange of momentum between the fluid layers induced by the vertical components of the transverse circulations and the second redistribution is caused by

the horizontal momentum exchange along the width of the channel caused by the transverse velocity component of the current.

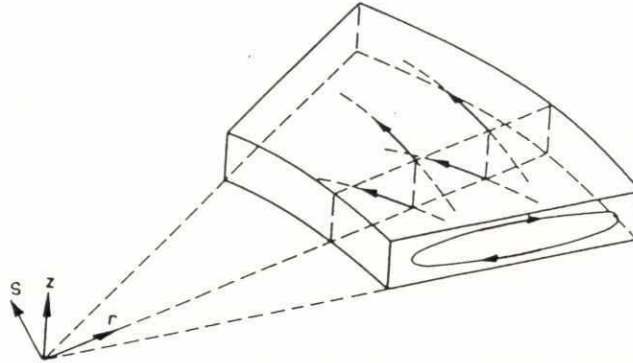


Fig. 2.2-3: Helical flow in river bend

The net influence of the secondary flow is an increase of the main flow velocity in the outer bend and a decrease in the inner bend.

The various distributions of velocity in 3 dimensions are shown in Fig. 2.2-4.

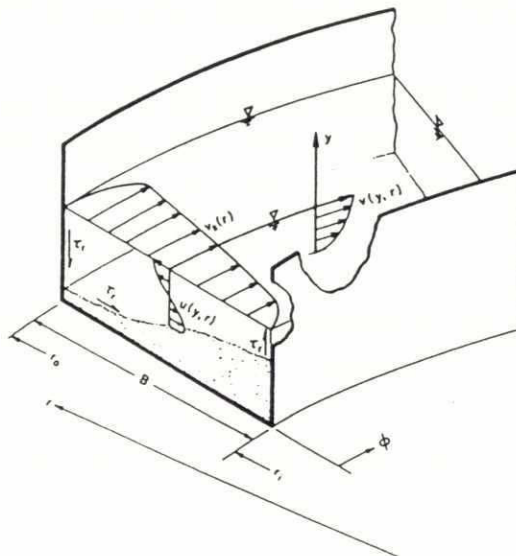


Fig. 2.2-4: 3-D sketch of velocity distribution in bend

d. Change of bed topography

The presence of the secondary current in a channel induces a lateral movement of the bed material in the direction of the lateral circulation in addition to the regular longitudinal transport. Also transverse and longitudinal bed slopes have significant influence on sediment transport. Erosion takes place in areas where water from a higher layer descends to the bed and the velocity of flow is high. Deposition takes place where the bottom water rises with low velocity to the upper layer. This induces the greater depths at the concave side of a channel bend and shoal depths at the convex bank. As a result the cross slope becomes steeper so that the perpendicular transport component is compensated by a transport downward due to gravity. The change of the bed level implies that in a natural river, equation (2.2-1) will not hold any more. The velocity U_s is relatively greater than in the outer bend as represented by equation (2.2-2).

$$U_s = R^{-1} \quad (2.2-2)$$

in which U_s is the velocity component in flow direction.

2.2.3 Description of Flow in Bends

Flows in alluvial channel bends are among the more difficult subject encountered by hydraulic engineers. The main difficulty encountered is the description of the helical flow pattern in bends and its interaction with the topography of the channel bed.

The centripetal acceleration of the water particles in a channel bend is created by a transverse slope of the water surface, where the level of the water surface at the inner bank is lower than the water surface at the outer bank. This implies that the centripetal acceleration is uniformly distributed over the vertical (hydrostatic pressure distribution).

As the friction at the bottom of the channel causes the longitudinal flow velocity to vary from a minimum near the bed to a maximum near the water surface, the flow particles close to the bottom will follow a path with a smaller radius of curvature than the flow particles closer to the water surfaces, which will cause the so-called helical motion.

This helical flow pattern can be considered as the combination of a longitudinal flow component (main flow) and a circulation in a plane perpendicular to the main flow direction usually called the secondary current. The secondary current is directed towards the centre of curvature near the bottom and directed outwards in the upper part of the cross-section.

The secondary flow has a significant influence on the main flow distribution due to convection of (main flow) momentum. In the central part of the flow horizontal convection is dominant. The net influence of secondary flow convection is an increase of the main flow velocity in the outer bend and a decrease in the inner part.

Steady incompressible turbulent flow can be described by four partial differential equations in curvi-linear co-ordinate system as shown in Fig. 2.2-5.

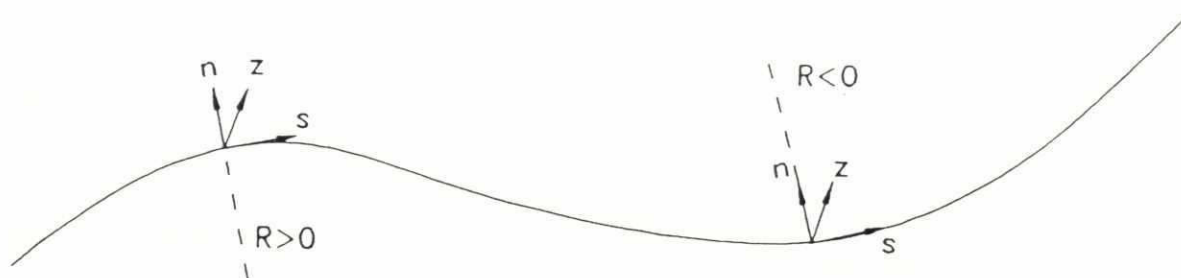


Fig. 2.2-5: Curvi-linear co-ordinate system of flow

According to Rozowskii (1957) the equations read as:

$$U \frac{\partial U}{\partial S} + V \frac{\partial U}{\partial n} + W \frac{\partial U}{\partial Z} + \frac{UV}{R} + \frac{1}{\rho} \frac{\partial P}{\partial S} = F_s \quad (2.2-3)$$

$$U \frac{\partial V}{\partial S} + V \frac{\partial V}{\partial n} + W \frac{\partial V}{\partial Z} - \frac{U^2}{R} + \frac{1}{\rho} \frac{\partial P}{\partial n} = F_n \quad (2.2-4)$$

$$U \frac{\partial W}{\partial S} + V \frac{\partial W}{\partial n} + W \frac{\partial W}{\partial Z} + g + \frac{1}{\rho} \frac{\partial P}{\partial Z} = F_z \quad (2.2-5)$$

$$\frac{\partial U}{\partial S} + \frac{\partial V}{\partial n} + \frac{\partial W}{\partial Z} + \frac{v}{R} = 0 \quad (2.2-6)$$

in which

g = acceleration due to the gravity

P = pressure

U, V, W = velocity components in S, n and Z directions, respectively

F_s, F_n, F_z = friction terms in S, n and z direction

ρ = density of water

The equations (2.2-3), (2.2-4) and (2.2-5) are three dynamical (momentum) equations (one for each direction) and the equation (2.2-6) represents the conservation of mass (continuity equation).



2.2.4 Description of Sediment Transport and Bed Deformation in Bends

Traditionally, one can identify three types of sediment transport: bed load, suspended load and wash load. Bed load is the part of sediment transport which is sliding, rolling, jumping or saltating over the bed. Suspended load is the part of sediment transport which for "some time" is carried in suspension due to turbulent mixing of the flow. The definitions of bed load and suspended load do not allow a strict distinction between these two types of sediment transport. Wash load is defined as transported material finer than the bed material, and depends not only on the local flow conditions and bed material, but also on the upstream conditions, hence it is difficult to predict. Usually, in alluvial rivers the morphological influence of wash load is neglected. There are several sediment transport formulas which can be used to predict bed load, suspended load or total load, such as: Kalinske (1947), Meyer-Peter-Muller (1948), Einstein (1950), Garde-Albertson (1961), Engelund-Hansen (1967), Ackers and White (1973) and van Rijn (1984).

Most bed load transport formulas consist of a function between two dimensionless parameters, as follows:

$$\text{- transport parameter} \quad X = \frac{S}{D^{3/2} \sqrt{\Delta g}} \quad (2.2-7)$$

$$\text{- flow parameter} \quad Y = \frac{\Delta D}{\mu h i} \quad (2.2-8)$$

in which

S = bed load transport per unit width

D = mean grain size

μ = ripple factor = $(C/C_{\text{grain}})^{3/2}$

C = Chézy coefficient

Δ = relative density of sediment, defined via $\Delta = (\rho_s - \rho)/\rho$

ρ_s = density of sediment

h = water depth

i = energy slope

The interaction between the bed load and the alluvial bed is one of the most fundamental aspects of the morphological processes in a river bend (Struiksmā et al., 1985 and Engelund, 1974). Bed deformation in bends and at crossings in alluvial rivers is an important factor for river engineering works in connection with navigability and bank stability. In addition, it is an intriguing feature in geomorphology since it governs, to a high degree, the planform of a river. An important class of rivers exhibit a relatively stable meandering planform. The main characteristic of the bed deformation in these rivers is the point bar in the inner bend and the pool in the outer bend.

It is reasonable to suppose that the lateral transport of bed load in a bend is determined by the direction of the flow at the bottom and the bed slope in the radial direction. At the initial stage of scour, where the bed is assumed to be horizontal, a sand particle will move along the same direction as the flow. However, as the scouring and deposition develops, the path of a sand particle will deviate from the direction of the flow due to the effect of lateral bed slope and, finally, it will take a concentric course for an infinitely long and circular channel. This final state is said to be stable state under hydraulic conditions, and is called "axisymmetric" condition.

Once the sediment transport has been formulated, based on the sediment and flow parameters, equations (2.2-7) and (2.2-8), the bed topography in the curvilinear co-ordinate system follows from the continuity equations for sediment.

$$\frac{\partial Z_b}{\partial t} + \frac{\partial S_s}{\partial S} + \frac{S_s}{R_n} + \frac{\partial S_n}{\partial n} + \frac{S_n}{R_s} = 0 \quad (2.2-9)$$

in which

Z_b = bed level above a reference level

S_s, S_n = the volumetric sediment transport flux included pores in s- and n- direction, respectively

R_s, R_n = radius of streamline and normal line curvature.

t = time co-ordinate

Many researchers have attempted to explain the bed deformation in meandering rivers and they have developed predictors for the point bar height and pool depth. A review on this subject (for axisymmetric conditions) is given by Odgaard (1981). Most of these models are based on the assumption that the lateral bed slope is determined by local parameters such as water depth, bed shear stress, bend curvature and sediment properties, i.e. by the balance between the upslope drag force induced by the spiral flow in a bend and the downslope gravitational force, both acting on the grains moving along the bed. Odgaard concludes that all the predictors which relate the local transverse bed slope to mean flow characteristics predict the slope to be proportional to the ratio h/R , expressed as:

$$i_T = \frac{d(h)}{dR} = K \frac{h}{R} \quad (2.2-10)$$

in which

i_T = transverse bed slope at the centre line.

K = ratio of transverse slope to h/r

The direction of the sediment transport (α) does not coincide with the direction of the bed shear stress (δ). This is due to the gravity force acting on the grains moving along a sloping bed. The effect is incorporated in the following formula by van Bendegom (1947) and also Koch and Flokstra (1980):

$$\tan \alpha = \frac{\sin \delta - \frac{1}{f_s \theta} \frac{\partial z_b}{\partial y}}{\cos \delta - \frac{1}{f_s \theta} \frac{\partial z_b}{\partial x}} \quad (2.2-11)$$

in which

f_s = a shape factor of the grains, usually with a value between 1 and 2.

θ = the Shields parameter, defined by $\theta = \frac{U^2 + V^2}{C^2 \Delta D_{50}}$ (2.2-12)

δ = the direction of the bed shear stress, which includes the effect of spiral flow, according to Koch and Flokstra (1980):

$$\delta = \arctan \left[\frac{v}{u} \right] - \arctan \left[A \frac{h}{R_*} \right] \quad (2.2-13)$$

$$A = 2 \epsilon \kappa^{-2} (1 - \sqrt{g/\kappa} C) \quad (2.2-14)$$

in which

A = coefficient which weighs the influence of spiral flow and depends on the vertical distribution, in the case of a logarithmic profile:

κ = von Karman constant, usually equals 0.4

ϵ = calibration coefficient

Furthermore the effective local radius of curvature of the streamline R_* is introduced, which is defined by:

$$R_* = \frac{H(u^2 + v^2)^{1/2}}{I} \quad (2.2-15)$$

in which

I = a measure of the intensity of the secondary flow, which can be derived from:

$$\beta \frac{c}{\sqrt{g}} h \frac{\partial I}{\partial S} + I = \frac{h}{R} \sqrt{u^2 + v^2} \quad (2.2-16)$$

in which

β = a given coefficient

R = the local radius of curvature of the stream line

2.2.5 Prediction of Axisymmetric Bed Topography in Bend

It is necessary to determine the bed topography or the steady state transverse bed slope in bend that can predict the maximum scour depth at the outer bend, in particular for bank

protection works. As mentioned in Section (2.2.3), several analytical models have been developed for the prediction of the steady state transverse bed slope in a curved, erodible bed channel. Among them are the models by van Bendegom (1947), NEDECO (1951), Yen (1967), El-Khudairy (1970), Zimmermann and Kennedy (1978), Falcon (1979), Odgaard (1981, 1984) and Struiksmma (1988). Each one of the models is based on the radial balance of forces acting on a sediment particle moving along a radially inclined bed and all of them, except the Odgaard model, are based on the grains of bed load moving along the bed surface, whereas Odgaard considers the bed surface characteristics only. Here only the application of three models will be described: the models of (i) Zimmermann and Kennedy, (ii) Odgaard and (iii) Struiksmma.

(1) The Zimmermann - Kennedy model

Zimmermann and Kennedy performed experiments in three circular plan formed channels to verify their analytical results.

They found the following equation for the transverse bed slope of the cross-section of a bend:

$$i_T = \alpha \frac{n + 1}{2n(n + 2)} \cdot \frac{h}{R_c} F_D^2 \quad (2.2-17)$$

in which

α = the ratio of projected surface area to volume for a non-spherical particle divided by that for a sphere (≈ 1.27)

n = velocity profile exponent, related to the Darcy-Weisbach friction factor and Chézy coefficient, defined by:

$$n = k \sqrt{\frac{8}{f}} = \frac{\kappa C}{\sqrt{g}} \quad (2.2-18)$$

in which

f = Darcy-Weisbach friction factor

F_D = the particle densimetric Froude number defined by:

$$F_D = \frac{\bar{U}}{\sqrt{\Delta g D}} \quad (2.2-19)$$

By using i_T in the equation (2.2-20), the maximum depth at the outer bend can be determined.

$$h_b = h + i_T (R_o - R_c) = h + \frac{Bi_T}{2} \quad (2.2-20)$$

in which

B = the channel width

o, c are the subscripts to represent outer bank and centre line of bend

(2) The Odgaard models

The procedure for the application of the Odgaard model (1981) is summarized as follows: given the cross sectional average values of depth, $\bar{h}$ and velocity, $\bar{U}$, and the cross-sectional average value of D_{50} , the values of n_f and N_f can be determined by equations (2.2-21) and (2.2-22)

$$n_f = \kappa F_{DC} \cdot \theta^{-\frac{1}{2}} \quad (2.2-21)$$

$$N_f = \frac{n_f + 1}{n_f(n_f + 2)} \quad (2.2-22)$$

in which

θ = dimensionless Shields parameter

F_{DC} = the particle Froude number based on V_{cr} and D_{cr} , that is:

$$F_{DC} = \frac{\bar{V}_{cr}}{\sqrt{\Delta \cdot g D_{cr}}} \quad (2.2-23)$$

in which

$\bar{V}_{cr}$ = critical mean flow velocity

D_{cr} = diameter of the particle whose motion is impending.

Then the factor of proportionality K can be estimated by equation (2.2-24) or (2.2-25).

$$K = \frac{3\alpha}{2} \theta \kappa^{-2} n_f^2 N_f \quad (2.2-24)$$

$$K = \frac{3\alpha}{2} N_f F_{DC}^2 \quad (2.2-25)$$

in which

α = ratio of projected surface area to volume for a sediment particle divided by that for a sphere (≈ 1.27).

Knowing the radius of curvature at the channel section, $\sin \beta$ can be obtained

$$\sin \beta = K \frac{h}{R} \quad (2.2-26)$$

in which

$\sin \beta$ = radial or transverse slope of the bed surface.

However, the field data indicate that a reasonable first approximation may be obtained by considering $\sin \beta$ to be constant across a section. In that case, the transverse depth variation may be described by the equation:

$$\frac{h_b}{h_c} = 1 + K \left[\frac{R}{R_c} - 1 \right] \quad (2.2-27)$$

in which subscript c is related to the channel centre.

In 1984, Odgaard introduced a new model for finding a transverse bed slope by the following equation:

$$i_T = \frac{3\alpha}{2} \frac{\sqrt{\theta_{cr}}}{\kappa} \cdot \frac{m+1}{m+2} \cdot F_D \cdot \frac{h}{r} \quad (2.2-28)$$

in which

i_T = transverse bed slope

α = 1.27 for ordinary river sand

θ_{cr} = dimensionless critical Shields factor which is substantially constant at a value of 0.06 for a region of fully developed turbulence, $R_{*c} > 400$

F_D = particle Froude number defined by equation (2.2-19)

R_{*c} = critical boundary Reynolds number

m = friction parameter (velocity profile exponent), related to the shear velocity (U_*), Darcy-Weisbach's friction factor (f) and Chézy coefficient (C), as

$$m = \bar{\kappa} U / U_* = \kappa \sqrt{8/f} = \kappa C / \sqrt{g} \quad (\text{Zimmermann and Kennedy 1978})$$

in which, κ = Von Karman's constant ($\kappa \approx 0.4$) and

$$(m + 1)/(m + 2) \approx 1$$

By assumption of laterally constant transverse slope over the central portion of the channel cross-section, the transverse depth distribution is given (in terms of centreline values) by:

$$\frac{h_b}{h_c} = 1 + i_{Tc} \frac{R_c}{h_c} \left(\frac{R}{R_c} - 1 \right) \quad (2.2-29)$$

in which

i_{Tc} = transverse bed slope at the channel centre line is given by equation (2.2-28).

(3) Struiksmas method

In 1988, Struiksmas constructed a graph for first estimation of the maximum scour depth at the outer bend of curved channels in terms of cross sectional average parameters.

The construction was based on the following equations (2.2-30) and (2.2-31) for the transverse bed slope and the transverse velocity distribution, respectively, valid for infinite long circular bends:

$$\frac{\partial h}{\partial v} = A \sqrt{\theta} \cdot \frac{h}{R} \quad (2.2-30)$$

$$\frac{U}{U_c} = \frac{\sqrt{h}}{h_c} \cdot \frac{R_c}{R} \quad (2.2-31)$$

in which

A = secondary flow coefficient based on Chézy coefficient:

$$A = 2\kappa^{-2} \left(1 - \frac{\sqrt{g}}{\kappa C}\right) * 0.85 \quad (2.2-32)$$

in which, C is related to subscript for centre line values.

In order to express the bend scour depth in terms of cross sectional averages, he used an integral equation for determination of the mean water depth, $\bar{h}$:

$$\bar{h} = \frac{1}{B} \int_{R_c - \frac{1}{2}B}^{R_c + \frac{1}{2}B} h dR \quad (2.2-33)$$

Struiksma defined the mean Shields parameter $\bar{\theta}$ as:

$$\bar{\theta} = \theta_c \sqrt{\frac{\bar{h}}{h_c}} \quad (2.2-34)$$

After integration of equations (2.2-30) and (2.2-33), he prepared the graph as shown in Fig.

(2.2-7) based on the definitions of Fig. (2.2-6). So it is possible to find the ratio $\frac{h_b}{h}$ (in

which h_b is the maximum bend scour depth) as a function of $A\sqrt{\bar{\theta}}$ for various values of $\frac{B}{R_c}$.

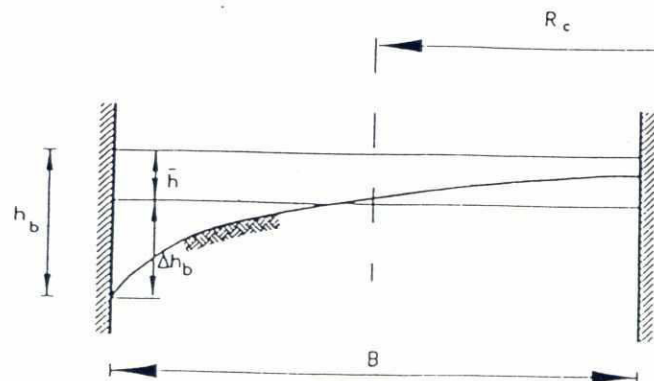


Fig. 2.2-6: Sketch of cross-section in bend

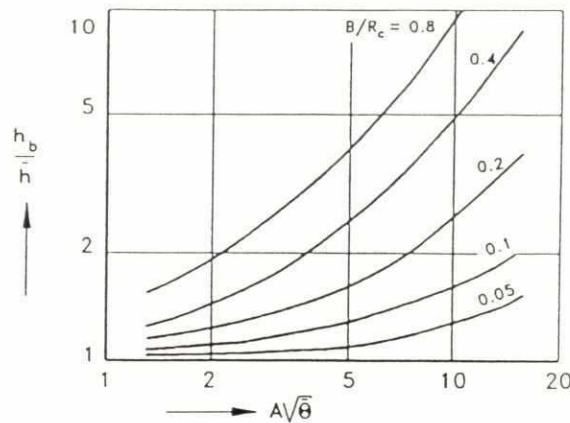


Fig. 2.2-7: Struiksma method for finding the maximum bend scour depth

2.2.6 Conditions in the Beginning of Bends

In Section 2.2.5 the transverse bed level profile is discussed for infinitely long bends and predictors for these conditions (often referred to as axisymmetric bend conditions) are also provided. For an infinitely long bend the transverse bed profile is no longer dependent on the conditions upstream of the bend. In the tests to be discussed in the next chapter these upstream conditions were a fairly uniform distribution of flow and sediment over the cross-sections. Adaptation to the conditions in a bend, where for the axisymmetric solution most of the water and the sediment is transported along the outer bend (see Section 2.2.5), takes some distance. In the experiments to be described later but also in real bends this adaptation was there and it is usually very pronounced. In the interpretation of the results it is therefore important to understand the nature of this adaptation to axisymmetric bend conditions. In this section some key elements of this adaptation are discussed.

The conditions in the beginning of a bend have been studied by in particular Struiksma et al. (1985), see also de Vriend and Struiksma (1983) and Struiksma and Crosato (1989). The equations to be used for an analysis of this more complete case, where also the phenomena in the beginning of a bend are included, are:

- (1) 2-dimensional depth averaged momentum equations for steady shallow flows, including the effect of momentum transfer due to the secondary flow;
- (2) continuity equation for the water in 2-directions;
- (3) expression for the direction of the bed shear stress which includes the effect of the spiral motion;
- (4) sediment transport formula;
- (5) expression for the direction of the sediment transport including the effect of the direction of the shear stress and the longitudinal and transverse slope of the river bed, and
- (6) sediment continuity equation in 2-directions.

To get some insight into what is happening in the beginning of bends, Struiksma et al., (1985) introduced some simplified approach. In their analysis, which is based on a linear analysis of the underlying equations, it was shown that the conditions in the beginning of a bend are determined by three parameters, notably λ_w , λ_s and b . In addition the axisymmetric conditions (and of course the upstream conditions) should be known.

The parameters λ_w , λ_s are defined via:

$$\lambda_w = \frac{C^2 h}{2g} \quad (2.2-35)$$

$$\lambda_s = \frac{1}{\pi^2} \frac{B^2}{h^2} f(\theta) h \quad (2.2-36)$$

in which

- C = Chézy coefficient ($m^{0.5}/s$)
- h = original water depth (m)
- g = acceleration due to gravity (m/s^2)
- B = width of river (m)
- $f(\theta)$ = function of θ
- θ = Shields parameter (-) defined via

$$\theta = \frac{hi}{\Delta D} \quad (2.2-37)$$

in which

- i = water level slope (-)
- Δ = specific density of sediment, defined via

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

in which

- ρ_s = density of sediment (kg/m^3)
- ρ_w = density of water (kg/m^3)
- D = characteristic particle size (mm). The function f of θ is at present assumed to read as:

$$f(\theta) = \frac{0.85}{E} \sqrt{\theta} \quad (2.2-38)$$

in which

- E = calibration coefficient (-), the value of which usually varies between 0.5 for flume tests and 1.0 for prototype conditions.

The parameter b is the power of the simplified sediment transport equation:

$$s = a u^b \quad (2.2-39)$$

in which

s = sediment transport per unit width (m^2/s)

a = coefficient (dimension dependent on value of b) and

u = flow velocity (m/s).

The parameters λ_w and λ_s are so-called relaxation lengths. To illustrate the concept of relaxation length, here an example will be considered that shows λ_w to be relaxation length for the flow. Consider a straight wide flume with a fixed horizontal bed, and axis x in downstream direction while the transverse direction is indicated by y . The wall friction is neglected. At $x = 0$ the velocity distribution is assumed not to be uniform but to vary over the width. This distribution can be described by:

$$u(0,y) = \bar{u} + u'(0,y) \quad (2.2-40)$$

where

$\bar{u}$ = average velocity over the width (m/s) and

u' = disturbance at the upstream end (m/s).

It may be assumed that u' is a function of both x and y . It is furthermore assumed that $u' \ll \bar{u}$. For large values of x the value of $u(x,y)$ will not depend on x anymore: the velocity is adapted to the bed and hence:

$$u(x \rightarrow \infty, y) = \bar{u} \quad (2.2-41)$$

See also Fig. 2.2-8 for an explanation.

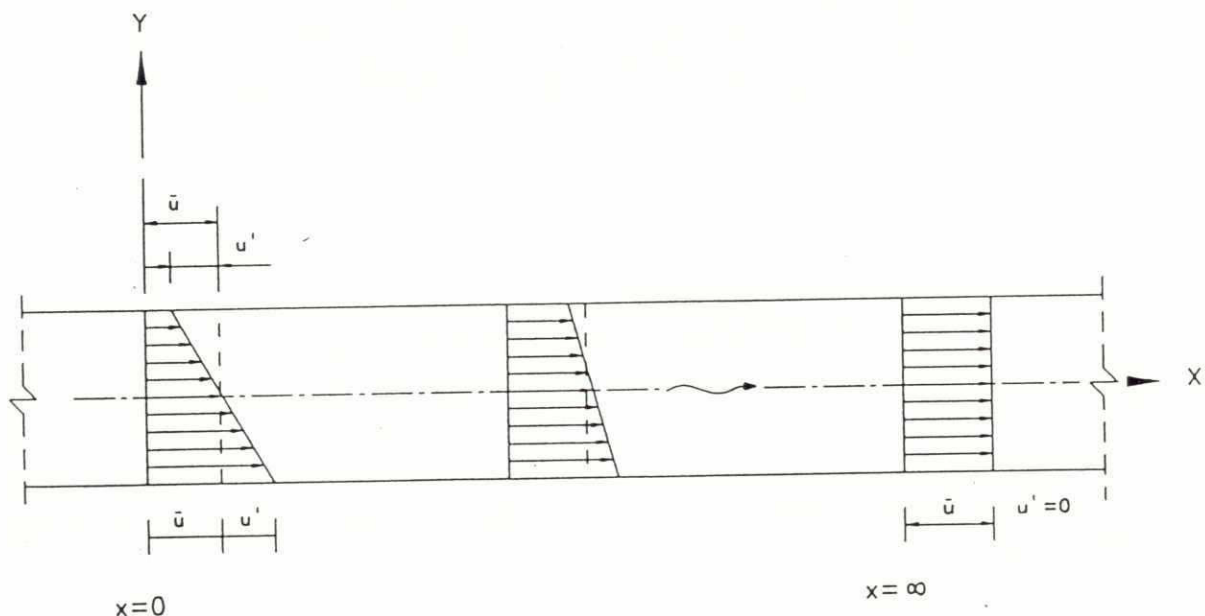


Fig. 2.2-8: Adaptation of flow to disturbance at $x = 0$

Here the interest is in the reduction of disturbance u' with distance. This can also be expressed as the reduction in vorticity, where the vorticity ω is defined as:

$$\omega = \frac{\delta(\bar{u} + u')}{\delta n} - \frac{(\bar{u} + u')}{R} \quad (2.2-42)$$

As the mean velocity is constant in n direction, and because $R \rightarrow \infty$, if $u' \ll \bar{u}$ this can also be written as:

$$\omega = \frac{\delta u'}{\delta n} \quad (2.2-43)$$

de Vriend and Struiksma (1983) have shown that

$$\omega(x) = \omega(0) \cdot e^{-x/\lambda_w} \quad (2.2-44)$$

Apparently the vorticity and hence the initial upstream disturbance u' , decreases in downstream position. The flow pattern adapts asymptotically to the equilibrium conditions. The rate of adaptation is characterized by the length scale λ_w , i.e. the distance along the streamline needed for the difference between $\omega(s)$ and $\omega(0)$ to be reduced by a factor e . This is schematically indicated in Fig. 2.2-9:

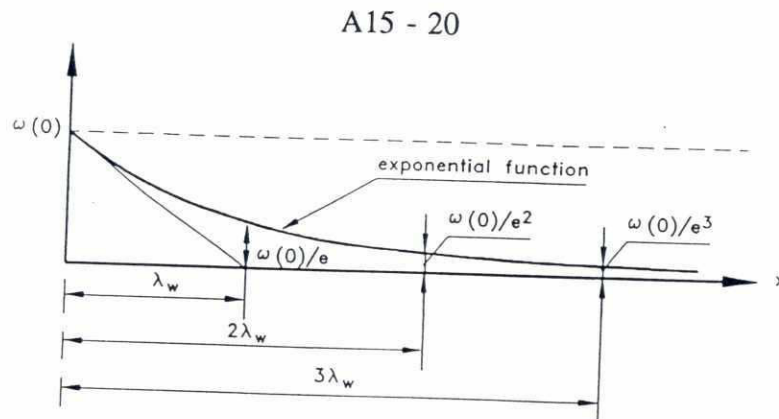


Fig. 2.2-9: Reduction in vorticity due to disturbance

This figure shows that λ_w can be seen as a characteristic adaptation length for the flow. The other parameter λ_s can, in a similar way, be considered as the adaptation of the bed level (and hence the sediment transport) to any new conditions. More details on the latter parameter are provided by van Mierlo (1986).

Going back to the conditions in the beginning of a bend, it was shown by De Vriend and Struiksma (1983), that the interaction between the flow and sediment transport can be described by a second order differential equation. The solution of this second order equation can be essentially different from the adaptation of the flow and the bed level. Depending on the values of the constants, and hence of the values of λ_w , λ_s and b , the solution can be

exponential or periodic, either growing or dampening. A possible solution in the periodic dampened range (which is characteristic for meandering rivers and occurred also in the flume experiments, see Section 5.2) is shown in Fig. 2.2-10.

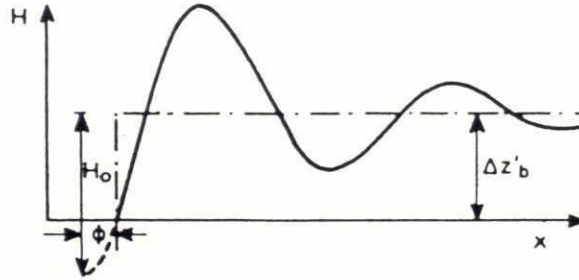


Fig. 2.2-10: Dampened periodic reaction to a change in bed level

This figure represents for example, the "reaction" of the river bed to a sudden step in the bed level, as happens in the beginning of a bend. The bed level in the outer bend tends to adapt to a new equilibrium, that corresponds to axisymmetric flow with a larger depth in the outer bend. The figure illustrates that the dynamic interaction between the transverse distribution between the flow and the sediment transport can give rise to a substantial "overshoot" of the transverse bed slope in the beginning of a bend. Within the periodic range the solution of the second order equation has the form of:

$$H = H_0 e^{-\frac{x'}{L_D}} \sin\left(\frac{2\pi}{L_p} x' + \phi\right) \quad (2.2-45)$$

in which

H_0 = (fictitious) amplitude of the oscillation at, $x' = 0$, i.e. the point where the step in "axisymmetric" bed level occurs, and

ϕ = distance lag (as represented in Fig. 2.2-10).

The wave number of the oscillation is given by:

$$\frac{2\pi}{L_p} = \frac{1}{2} \left[\frac{b+1}{\lambda_w \lambda_s} - \frac{1}{\lambda_s^2} - \frac{b-3}{2\lambda_w^2} \right]^{\frac{1}{2}} \quad (2.2-46)$$

and the length scale of the dampening by:

$$L_D = \frac{4\lambda_w \lambda_s}{2\lambda_w - (b-3)\lambda_s} \quad (2.2-47)$$

From the above expressions it can be derived that the values of L_p and L_D are fully determined by the values of λ_s/λ_w and the power b of the sediment transport equation. For $b = 5$ (corresponding to the Engelund-Hansen sediment transport equation) the solutions of these expressions are graphically indicated in Fig. 2.2-11. Fig. 2.2-12 provides the variation

of L_p and L_D with the power b , whereby b varied between 4 and 5. These values correspond to the values of b in the experiments (see Section 5.2).

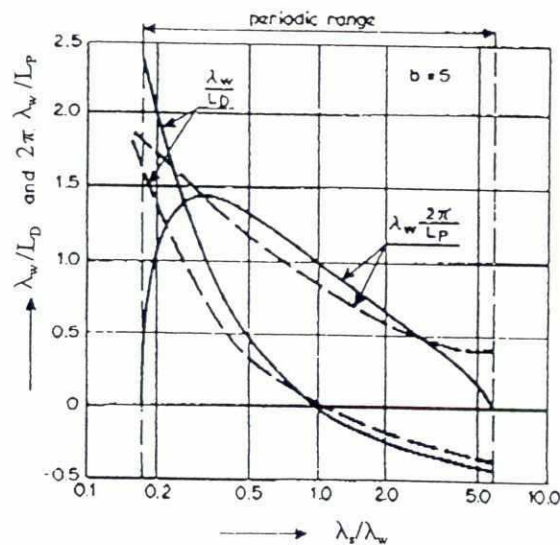


Fig. 2.2-11: Relative wave number and relative damping as a function of λ_s/λ_w for $b = 5$

The above "theory" is used in Section 5.2 for the interpretation of the experiments as far as they were carried out in the curved flume.

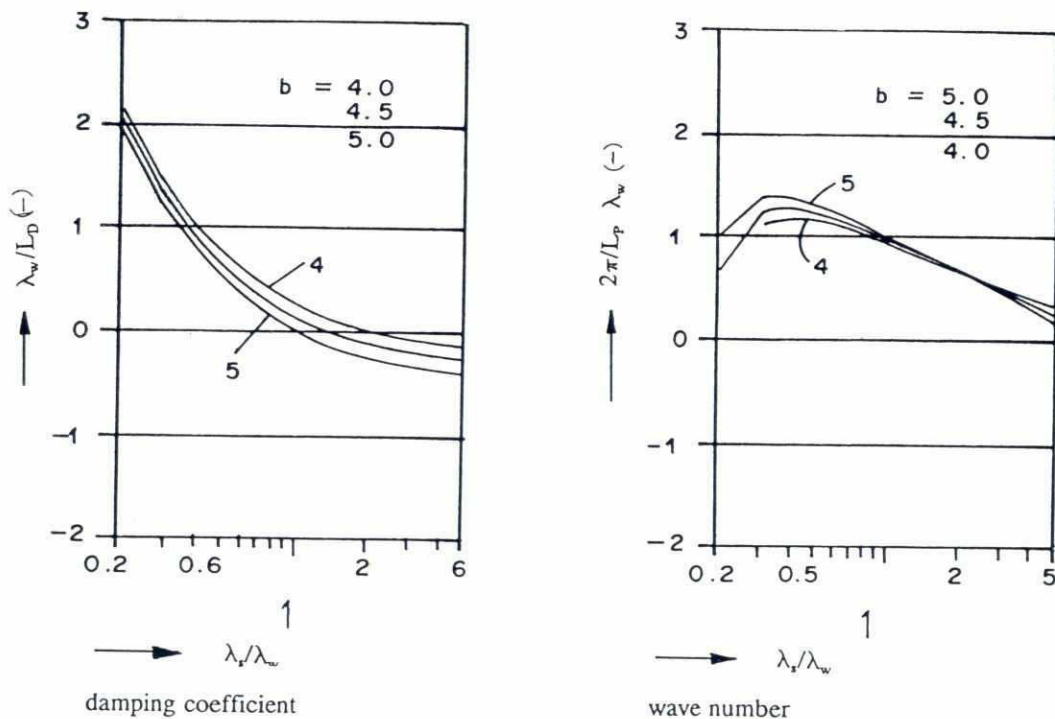


Fig. 2.2-12: Variations of the relative wave number and relative damping as a function of λ_s/λ_w due to changes of the power b

2.2.7 Effect of Bank Erosion Products on Bed Topography in Bends

2.2.7.1 General

There is not much literature available on the effects of bank erosion products on bed topography especially in bends. Most researchers have neglected this effect in their publications, in other words they have assumed that 100 % of bank erosion products goes into wash load and thus will not have any influence on the sediment balance ($\frac{\partial n}{\partial t} = 0$).

Banks usually consist of a fraction of cohesive materials which become wash load after erosion and do not affect the balance of sediment, and a fraction of granular materials, almost with the same properties of the bed material. Therefore, this part of bank erosion products participate in the sediment balance. Migration rate has been found to vary directly with sediment imbalance. The sediment imbalance is defined as the difference between the sediment transport capacity and the actual sediment flux.

River flow, bed topography and planform have interrelations and as a consequence they are all affected by changes in bank erodibility. River bank erosion is a complex phenomenon and are determined by flow, bed topography, sediment transport, bank material properties and water quality. The main appearances of bank erosion are river widening and meandering. They often occur together, thus their analysis becomes complicated.

In a river bank erosion the flow exerts shear stresses that can remove particles from the bank. The near bank flow pattern is determined by discharge, movement of ships and wind. Bank erosion consist of (i) lateral entrainment of lower parts of the bank and (ii) near bank bed degradation, both including mass failure of upper parts. The bed topography near the banks is important for bank erosion, indirectly by affecting the flow velocities and consequently shear stresses, and directly by determining the height of bank which is very important for the stability of banks. Groundwater flow can also have an important effect on bank erosion, especially during recession of river water level after flooding, but is also related to land use and precipitation.

Many bank material properties are important for the resistance to erosion, such as bank material weight and texture, shear strength and cohesive strength, physio-chemical properties, bank height and cross sectional shape, groundwater level and permeability stratigraphy, tension cracks, vegetation and constructions. One of the main results of bank erosion is the lateral and downwards migration of river. The input of bank erosion products appears to decrease the transverse bed slopes in curved rivers, but it hardly affects the wave lengths and damping lengths of flow and bed topography in rivers with banks, which migrate only moderately (Mosselman, 1989). However, Murshed (1991) concluded that in more extreme cases bank erosion products increase the damping of the system of channel and can either increase or decrease the wavelength, depending on the particulars of the rivers concerned.

2.2.7.2 Bank Erosion Equations

Shear stresses exerted by the flow may cause erosion at the toe of the bank, which can be subdivided into lateral fluvial entrainment, Δn , and near bank bed degradation, Δz_b . Both types of erosion may induce mass failure, as they decrease bank stability, see Fig. 2.2-13.

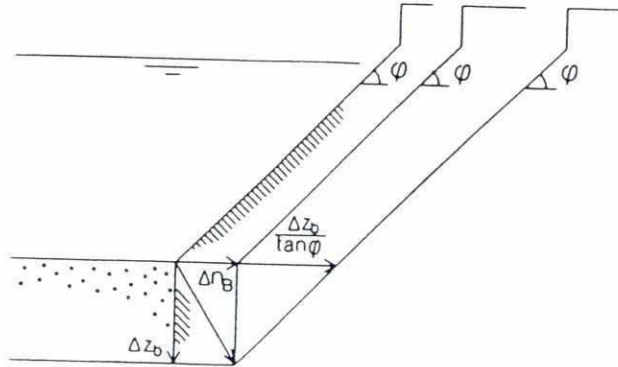


Fig. 2.2-13: River bank erosion due to lateral fluvial entrainment, Δn , and near bank bed degradation, Δz_b

Bed degradation results from gradients in sediment transport capacity which can be determined from the flow field by using an appropriate sediment transport formula. The lateral erosion can be determined with a simple but generally used relation for the erosion of cohesive soils which reads as:

$$\frac{\partial n}{\partial t} = E \left[\frac{\tau_{\text{bank}}}{\tau_c} - 1 \right] \quad \text{for } \tau_{\text{bank}} \geq \tau_c \quad (2.2-48)$$

$$\frac{\partial n}{\partial t} = 0 \quad \text{for } \tau_{\text{bank}} < \tau_c \quad (2.2-49)$$

in which

$$\frac{\partial n}{\partial t} = \text{erosion rate.}$$

$$E = \text{erodibility coefficient}$$

$$\tau_{\text{bank}} = \text{flow shear stress on the bank}$$

$$\tau_c = \text{critical shear stress below which no erosion occurs.}$$

On the other hand, the flow induced rate of bank erosion is assumed to be proportional to the near bank tangential velocity excess from the reach-averaged value (similar to Ikeda et al., 1981). It is assumed that bank erosion caused by bank instability is proportional to the near bank water depth deformation (with respect to the reach-averaged value). So combining the effects of velocity and water depth deformation, the erosion law can be obtained as follows:

$$\frac{\partial n}{\partial t} = E_U U + E_h H \quad (2.2-50)$$

in which

U, H = near bank velocity and water depth deformation with respect to their reach-averaged values.

E_U, E_h = time-averaged erosion coefficients.

In equation (2.2-50) the coefficient E_U is dimensionless, but E_h is not dimensionless. This equation can be expressed in a more elegant way by adopting the following equation, in which both the coefficients are dimensionless:

$$\frac{\partial n}{\partial t} = E_U U + E_h \frac{U_o}{h_o} H \quad (2.2-51)$$

The banks are assumed to consist of a fraction (ω) of cohesive material which becomes wash load after being eroded, and a fraction $(1-\omega)$ of granular material, with the same properties of the river bed material. Hence, the volume of bank erosion products per unit length of river, ΔV_1 , to be accounted for in the sediment balance after retreat due to lateral erosion, Δn , can be estimated as:

$$\Delta V_1 = (1-\omega) H \Delta n \quad \text{for } \Delta n > 0 \quad (2.2-52)$$

in which

H = total bank height

Also, the volume of bank erosion products per unit length of river, ΔV_2 , to be accounted for in the sediment balance after bed degradation and ΔZ_b can be estimated as:

$$\Delta V_2 = (1-\omega) H \frac{-\Delta Z_b}{\tan \varphi} \quad \text{for } \Delta Z_b \leq 0 \quad (2.2-53)$$

in which

φ = bank slope.

Although the value of φ is variable during bed degradation, it is assumed constant, and Δn is taken positive for both left and right banks, whereas erosion of bed corresponds to a negative value of ΔZ_b .

The sediment balance reads:

$$\frac{\partial Z_b}{\partial t} + \frac{\partial S_x}{\partial x} + \frac{\partial S_y}{\partial y} = 0 \quad (2.2-54)$$



in which

t = time

S_x, S_y = sediment transport rates per unit width in longitudinal and transverse direction, respectively.

The transverse sediment transport rate, S_y , is made up of various contributions:

- transport due to transverse component of the bed-shear stress exerted by the flow;
- transport due to gravity acting along a sloping bed, and
- transport due to lateral input of bank erosion products.

The direction of the bed shear stress differs from the depth-averaged flow direction due to the influence of secondary flow. The equation for the transverse sediment rate can be written as:

$$S_y = S_x \frac{U}{V} - S_x A \frac{h}{R_*} S_x \frac{1}{f(\theta)} \frac{\partial Z_b}{\partial y} + S_{bank} \quad (2.2-55)$$

in which

$f(\theta)$ = function which weighs the influence of the transverse bed slope

S_{bank} = transverse transport of bank erosion products.

For the first three right hand terms of equation (2.2-55) reference is made to Koch and Flokstra (1980).

2.2.7.3 Influence on Transverse Bed Slope

Finally, the influence of bank erosion products on the transverse bed slope can be expressed in an axisymmetric case by the following equation derived by Mosselman, 1989:

$$h' = Af(\theta_o) \frac{B_o H_o}{\pi R_c} - \lambda_s \frac{h_o \mu_o h_o}{B_o S_{xo}} \frac{\partial n}{\partial t} \quad (2.2-56)$$

in which

R_c = radius of curvature of channel centre line

λ_s = adaptation length of secondary flow

μ_o = relative effective bank height expressed by

$$\mu = \frac{(1-\omega)H}{h} \quad (2.2-57)$$

h' = first-order perturbation of h , varying with S only

o = subscript is related to zero order solution

By using the definition of λ_s ; i.e.;

$$\lambda_s = \frac{1}{\pi^2} \frac{B_o^2}{h_o^2} f(\theta_o) h_o \quad (2.2-58)$$

The equation (2.2-56) can be change which reads:

$$\pi \frac{h'}{B_o} = f(\theta_o) \left[A \frac{h_o}{R_c} - \frac{1}{\pi} \frac{\eta_o H_o}{S_{xo}} \frac{\partial n}{\partial t} \right] \quad (2.2-59)$$

It is obvious that the input of bank erosion products reduces transverse bed slopes. In the derivations Mosselman has assumed that bank erosion products are distributed evenly over a cross-section. In reality their distribution will only extend over a part of the cross-section, so the actual reduction of transverse bed slopes can be expected to be stronger in regions near eroding banks. As those are the regions where the largest transverse slopes are found, it seems reasonable to expect that the overall effect will also be a stronger reduction than predicted by the above equations.

2.3 LOCAL SCOUR AROUND GROYNES

2.3.1 General

River training, in its broad aspects, covers all engineering works constructed on a river to guide and confine the flow to the river channel. These are done to control or regulate the river bed configuration for effective and safe movement of floods and river sediment and navigation. Based on the conditions and the aims, various river training works can be executed such as guide bunds, groynes or spur dikes, levees or embankments, cut-offs, etc.

Groynes or spur dikes are stone, gravel, rock, earth or pile structures built at an angle to a river bank to deflect flowing water away from critical zones, so as to prevent erosion of the bank, to establish a more desirable channel for flood control, navigation and erosion control. They are used in wide braided rivers to establish a well defined channel that neither aggrades and degrades nor shifts its location from year to year. Spurs are also used in meandering rivers to control flow into or out of a bend or through a crossing.

Groynes may be aligned either perpendicular to the bankline or at an angle pointing upstream or downstream. A groyne pointing upstream has the property of repelling the river flow away from it and is termed a repelling groyne. When the groyne has a short length, usually it changes only the direction of flow without repelling it. This kind of groyne is known as deflecting groyne and gives only local protection. On the contrary, a groyne pointing downstream attracts the river flow towards it, and is called an attracting groyne. Straight groynes and other types of groynes are represented in Fig. 2.3-1 and 2.3-2.

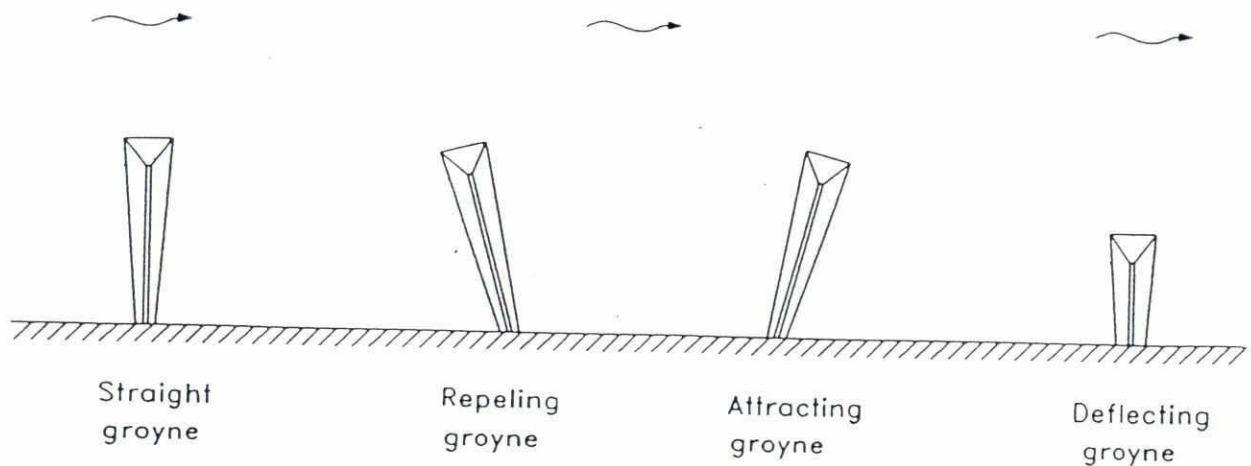


Fig. 2.3-1: Various types of straight groynes

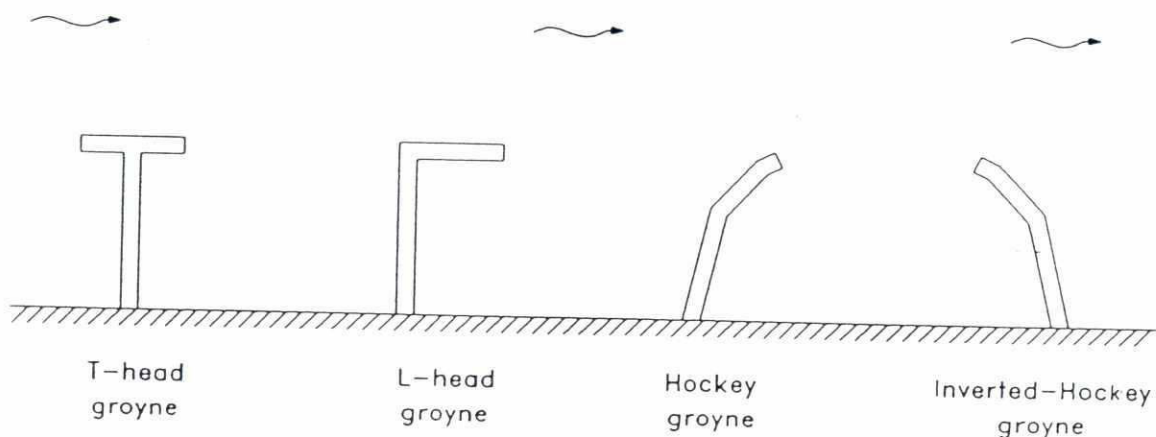


Fig. 2.3-2: Other types of groynes

2.3.2 Flow Pattern Near and Around Groynes

Flow constrictions, whether man made or natural, occur whenever there is a significant reduction in the width of the canal or river. The simplest case is the long contraction in alluvial channels. Another typical example is the diversion of a river course by construction of a spur dike or groyne either to protect the banks from erosion or to train the mainstream along a desired alignment.

Due to the flow concentration and the local disturbance to the flow in the vicinity of the structures, scouring of the alluvium within and around the constricted part usually occur. In hydraulic engineering, the extent of this erosion and the disturbance caused by these structures to the river flow is important.

The flow field around a spur dike is characterised by an acceleration from upstream to the most contracted cross-section somewhere in, or just downstream of the head of the groyne, followed by a deceleration of the flow. A separation point of the flow is located at the head of the spur dike. Downstream of the groyne, the main flow is separated from a large eddy by a vortex street. Downstream of the reattachment point normal flow will be established

again. In an extreme situation, just upstream from the spur dike a separation point and a small eddy can exist. Then, on the upstream side of the groyne there is a reattachment point, see Fig. 2.3-3; the down flow at the groyne with vertical walls can generate a strong spiral motion near the bed.

In general the stability of the not-submerged spur dike is endangered by a scour hole due to the strong vortices just downstream of the separation point at the head of the spur dike. A too steep scour hole can induce a sliding of a part of the head into the scour hole or undermining of this head.

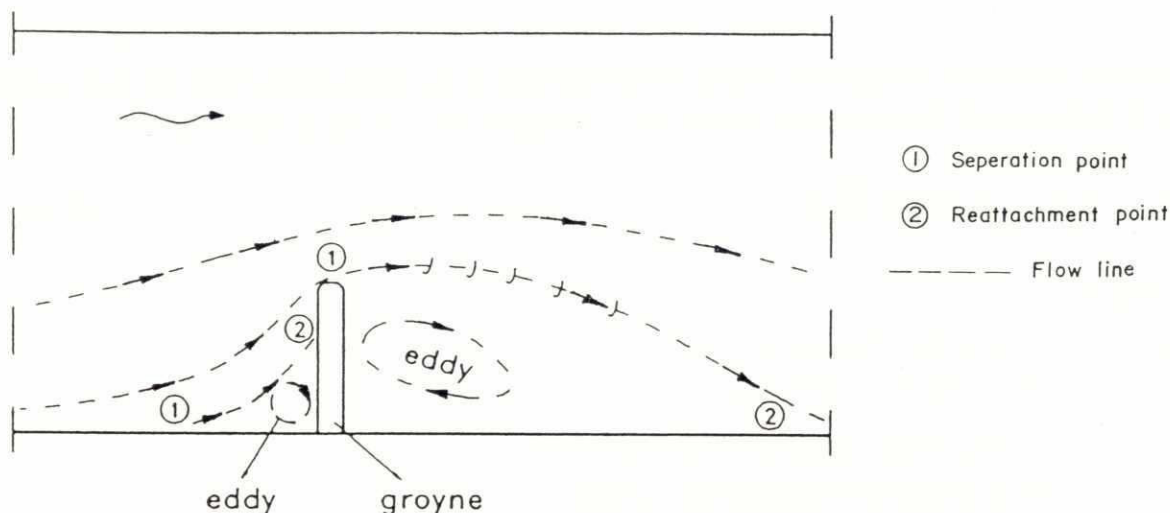


Fig. 2.3-3: Flow around a groyne

2.3.3 Prediction of Local Scour Depth Near Groynes in Straight Channels

The problem of scour around any obstruction placed in an alluvial channel is of great importance to hydraulic engineers. In practice, a channel is often obstructed by structures such as training dikes, bridge piers, abutments, groynes and so on. To be able to design a safe and economic structure, it is essential to have a clear picture of scour phenomena around them. All such structures concentrate river flow and introduce 3-D flow patterns and can therefore cause local scour.

As the water flows around the spur dike, the flow pattern is changed due to the reduction of width of channel, and as a result of this, the shear stress distribution around the groyne is changed. This leads to scouring action until equilibrium is established between the various forces influencing the scouring action. In other words, local scour around a spur dike results from acceleration of flow and vortex systems induced by obstruction to the flow. As the scour depth increases, vortex strength reduces, transport rate reduces and an equilibrium is established and finally scouring ceases. Although the vortex system is known to be the cause of local scour, it has not been possible as yet to calculate the strength of the vortex and relate

the velocity field with subsequent scour. In general, the stability of the not-submerged spur dike is endangered by a scourhole due to the strong vortices just downstream of the separation point at the head of the spur dike. A too steep scour hole can induce a sliding of a part of the head of groyne into the scour hole or can undermine this head.

Local scour depth varies with time because sediment transported into the scour hole varies. Sediment transport varies most with a dune bed configuration and large dunes produce large variation in scour depth.

Many researchers have worked on the various aspects of the hydraulic characteristics of groynes, especially about the estimation of the maximum local scour depth at the head, or just downstream of the groyne. Inglis (1949) compared the total scour depth ($h_o + \Delta h_1$) with the 3-dimensional Lacey regime depth and formulated the following equation:

$$h_o + \Delta h_1 = 0.47K \left(\frac{Q}{f} \right)^{0.33} \quad (2.3-1)$$

in which

h_o = uniform flow depth

Δh_1 = maximum scour depth

f = silt factor, equal to $1.76 \sqrt{D}$

D = diameter of bed material in mm.

K = factor for flow and geometric conditions which has some recommended values such as these in the following table:

Location of Scour	K (-)
Heads (1.5 V : 1 H)	3.8
Same with long sloping heads (1 V : 20H)	2.25
Scour at noses of large-radius guide banks	2.75
Scour at spurs along banks, depending on severity of attack	1.7-3.8

Table 2.3-1: Value of K as a function of the hydraulic and geometric conditions

Using Lacey's formula, the scour depth can be related to the discharge intensity by the following relation:

$$h_o + \Delta h_1 = 0.90 \left[\frac{q^{2/3}}{f^{1/3}} \right] \quad (2.3-2)$$

in which

q = the discharge per foot width of main channel

f = silt factor

Ahmad (1953) performed scouring studies on single spurs in vertically sided channels and he introduced this formula for the maximum scour depth:

$$h_o + \Delta h_1 = k(\alpha_1) \cdot q_c^{2/3} \quad (2.3-3)$$

in which

- α = spur dike angle
 $K(\alpha_1)$ = a factor as a function of α_1 which can be estimated from Table 2.3-2
 q_c = discharge intensity at the constricted section.

α_1	units	30°	45°	60°	90°	120°	150°
$K(\alpha_1)$	metric	1.8	2.0	2.15	2.25	2.4	2.45

Table 2.3-2: Values of K as a function of spur angle α_1

Liu, Chang and Skinner (1961) performed extensive studies on vertical wall type groynes and some abutments in two flumes, with $B=4.2$ m and 2.4 m. They found an empirical formula such as the one below:

$$\frac{d_{sc}}{h_o} = 2.15 \left[\frac{B_s}{h_o} \right]^{0.4} Fr^{1/3} \quad (2.3-4)$$

in which

- d_{sc} = scour depth in equilibrium
 Fr = Froude number, defined as $Fr = U_o/(g \cdot h_o)^{0.5}$
 B_s = width of a scour hole

The analysis of field data from spurs along the Mississippi river confirmed equation (2.3-4) and shows an upper limit of B_s/h_o , according to Richardson et al. (1975):

$$\frac{B_s}{h_o} \leq 25 : \text{equation (2.3-4)}$$

$$\frac{B_s}{h_o} > 25 : \text{equation (2.3-4) with } B_s/h_o = 25$$

$$\frac{h_{sc}}{h_o} = 7.8 Fr^{0.33} \quad (2.3-5)$$

For conditions with dunes, h_s fluctuates around the average value of the h_{sc} . The maximum value h_{sm} is found by adding $0.3h_o$ to h_{sc} .

Garde et al. (1961) conducted an extensive study on plate-form spur dikes in a 2 ft width flume and studied the effect of flow, spur dike and sediment characteristics. It was found that the Froude number of the normal channel, the opening ratio, the angle of inclination of the

groyne, and the averaged drag coefficient of the sediment particle adequately represent the influence of flow, spur dike and sediment characteristics on the maximum scour depth. They found the equation (2.3-6) which reads as:

$$\left[\frac{h + \Delta h_1}{h} \right] = k \frac{1}{\alpha} Fr^n \quad (2.3-6)$$

in which

K and n are functions of C_D (Fig. 2.3-4)

$$C_D = \frac{4}{3} \frac{\Delta_s D}{w^2 \rho} \quad (2.3-7)$$

in which

Δ_s = difference of specific weight between sediment and water

w = settling velocity's of sediment (Fig. 2.3-5)

and

$$\alpha = \text{opening ratio} = \left[\frac{B-b}{B} \right]$$

in which

B = width of the channel

b = width of the spur dike

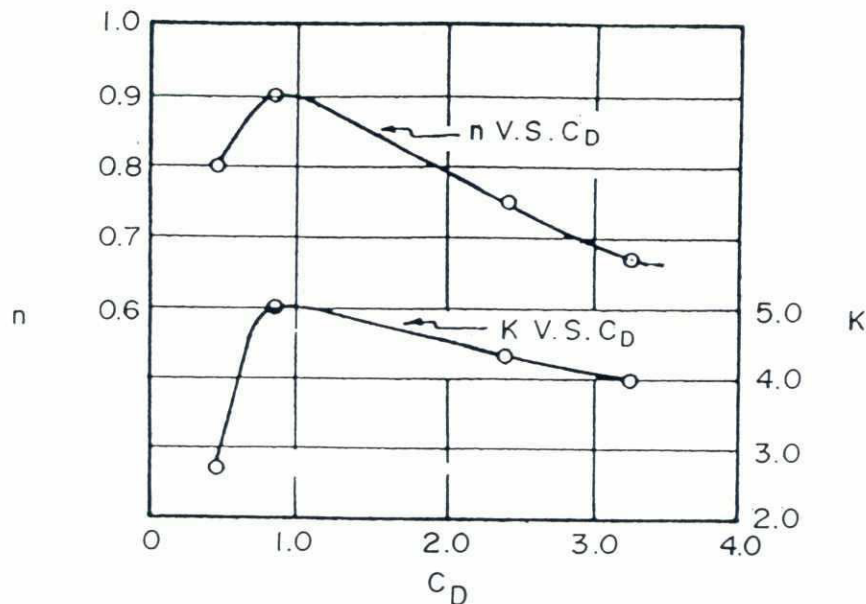


Fig. 2.3-4: Variation of n and K with C_D

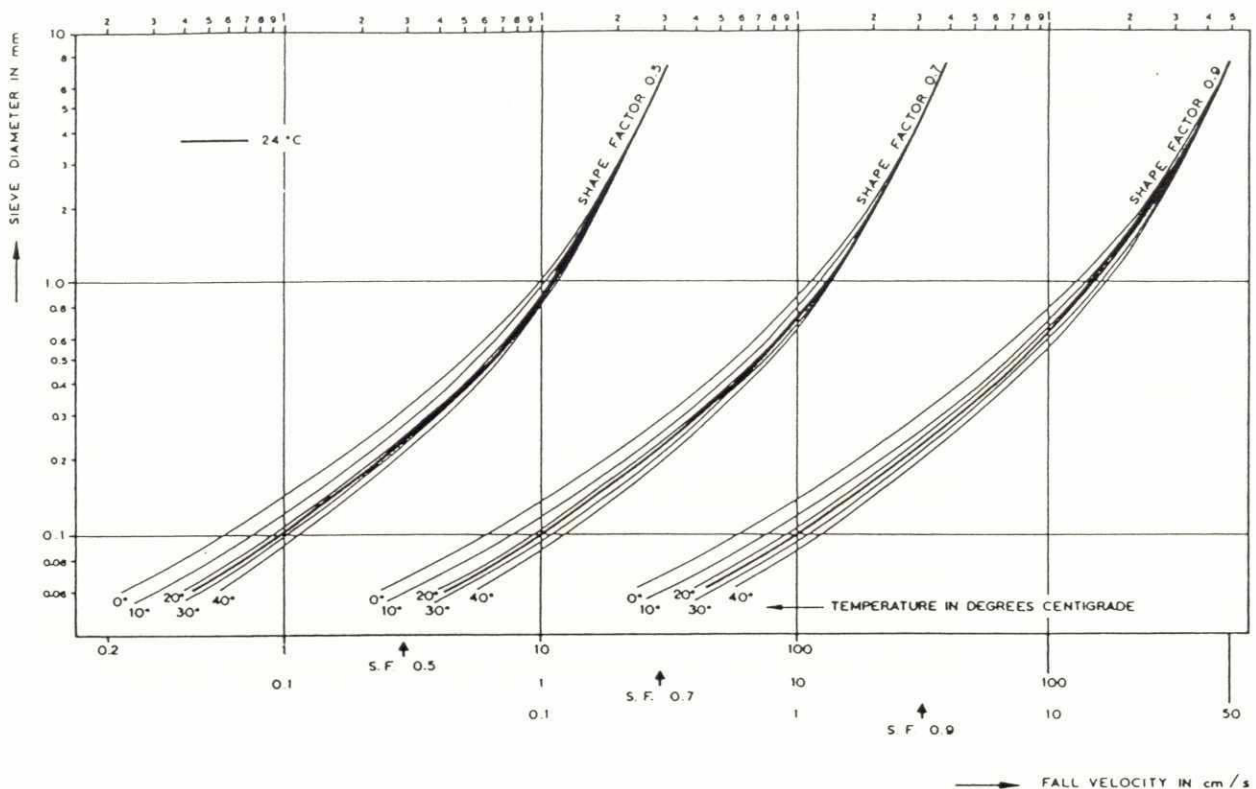


Fig. 2.3-5: Relation of sieve diameter and fall velocity for naturally worn quartz particles falling alone in quiescent distilled water of infinite extent

Gill (1972), conducted numerous experiments in a 2.5 feet wide flume to study the effect of grain size on bed material, flow depth and also the effect of involving bed load movement of flow on the depth of equilibrium scour around spur dikes (Fig. 2.3-6). He concluded that the depth of equilibrium scour is affected by the size of bed material and also the depth of uniform flow upstream of the groyne location. The combined effect of both of these have been empirically formulated as:

$$\alpha = 8.375 \left[\frac{D}{h_1} \right]^{0.25} \quad (2.3-8)$$

in which

h_1 = depth of flow at the upper uncontracted channel

Also he found that the depth of maximum scour occurs when the sand bed upstream of the spur is at the threshold of movement and for the purpose of modelling and basic research, it is necessary the distinction between clear water scour while the scour caused by bed load transporting flow should be considered. To determine the maximum depth of scour, Gill introduced the following formula:

$$\frac{h_{sl}}{h_1} = \alpha \left(\frac{B_1}{B_2} \right)^{6/7} \left[\frac{1}{\left(\frac{B_1}{B_2} \right)^{1/n} \left(1 - \frac{\tau_c}{\tau_1} \right) + \frac{\tau_c}{\tau_1}} \right]^{3/7} \quad (2.3-9)$$

$$1 - \frac{\tau_c}{\tau_1} = 0 \quad \text{for} \quad \frac{\tau_c}{\tau_1} \geq 1$$

in which

- α = a coefficient depending on the flow depth and the size of bed material defined by equation (2.3-8)
- τ_c = critical shear stress of the bed material (Fig. 2.3-7)
- τ_1 = bed-shear stress in the upper uncontracted channel
- n = a numerical exponent related to various well known bed load equations (for instance: $n=1.5$ related to the Meyer-Peter-Muller formula)

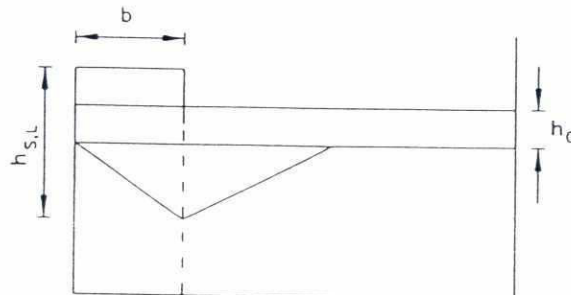


Fig. 2.3-6: The cross-section of the experimental flume of Gill

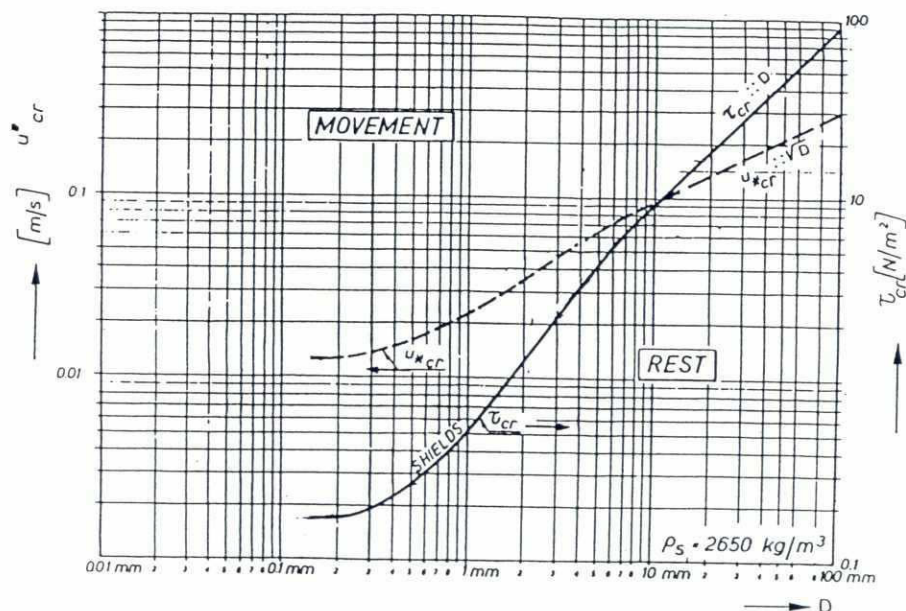


Fig. 2.3-7: Critical shear stress and critical shear velocity as function of grain size for $\rho_s = 2650 \text{ kg/m}^3$ (Sand)

Da Cunha (1971) performed a great number of tests with various vertical-side headed groynes and water depth in the range of 0.04 to 0.14 m with velocities between 1.0 and 1.4 U_c . He found the maximum scour depths around the head of the groyne from the following formula:

$$\frac{h_{sc}}{h_o} = 1.65 \left[\frac{b}{h_o} \right]^{0.3} \left[\frac{h}{h_o} \right]^{0.95} \left[\frac{w}{U_*} \right]^{0.15} \quad (2.3-10)$$

in which

U_* = shear velocity
 h_o = depth of uniform flow

The above mentioned relations derived from the various investigations on groyne scour models can be subdivided into four groups:

- those for which $U_o > U_c$, and which gives h_s as a function of the geometry of the obstruction;
- relations giving h_s as a function of flow velocity as well as the geometry of the obstruction;
- relations which consider the discharge intensity in the contracted flow section as well as geometrical properties, and
- relations which regard the effect of the bed material size as well as geometry properties.

The equations of type a and b have the disadvantage that they are derived from small scale experiments. But the equations of type c and d are more practical and are better applicable give accurate results for natural conditions.

2.4 COMBINED SCOUR

2.4.1 General

In Sections 2.2 and 2.3 the theory and mechanism of bend scour and local scour around the groynes were discussed and it is clear that many researchers have studied theoretically and experimentally these phenomena applying several methods. Formulas have been introduced for the determination of bed topography in bends and for the prediction of the maximum scour depth around the head of the groynes in straight channels.

When the groynes are used to protect the banks in bends or for other purposes, such as flow pattern maintenance, the scouring phenomenon around the groynes used at the outer bend becomes much more complex and it is difficult to analyse the scour hole which is affected by:

- spiral motion of the bend flow;

- eddies existing before and after the groyne;
- vortices in front of the groyne;
- constriction of the flow;
- transverse and longitudinal bed slope of the channel;
- shape, length and distance of the groynes, and
- bank erosion products.

Hence, there is a complex combined scour around the groynes constructed in bends, which up till now hardly has been investigated by researchers. The Project study could find the only research on this subject by Ahmed (1953), whose work will be discussed in Subsection 2.4.2 and as it was mentioned in Section (1.2), the main purpose of this study is to determine with experimental methods the maximum scour depth around groynes in the outer bend.

2.4.2 Regime Approach

As explained in Subsection 2.3.3 Ahmad (1953) performed his scouring studies on a single groyne in vertically-sided channel and as a basis for the total scour depth ($h_o + \Delta h_1$), he used a regime approach such as the dimensional Lacey regime method to formulate his equation (2.4-1) which reads as:

$$h_o + \Delta h_1 = k (\alpha_1) \cdot q_c^{2/3} \quad (2.4-1)$$

To demonstrate the effect of an unsymmetric velocity distribution, such as would occur in a bend, he performed several tests in a curved channel where the spurs were put in at positions I to IV successively (Fig. 2.4-1).

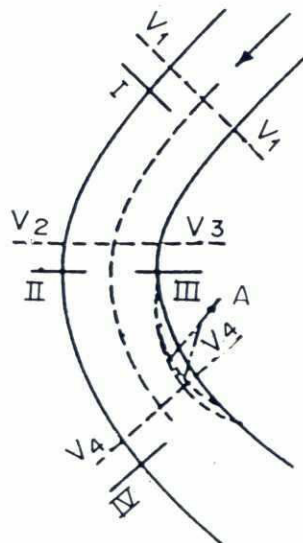


Fig. 2.4-1: Spurs in river bend, related to Ahmad's experiment

It was found that an unsymmetric velocity distribution increases the total scour depth roughly in proportion to the ratio of the velocity in the river section adjacent to the groyne and the average velocity. Ahmad recommends the following values for K (Table 2.4-1).

Spur Geometry	K (metric units)
spur below a bend on the concave side with swirl behind the bend	3.0 - 3.35
spur in moderate bend on swirl	2.2 - 2.60
spur in straight channel $\alpha_1 = 30^\circ - 90^\circ$	1.8 - 2.20
spur in straight channel $\alpha_1 = 90^\circ - 120^\circ$	2.2 - 2.60

Table 2.4-1: K as a function of the spur geometry

Ahmad did not clarify how to determine q_c in bends and how he calculated K based on $(h_o + \Delta h_1)/q_c^{2/3}$. It is clear that there is no uniform distribution of q in bends due to bed topography and velocity distribution.

3 APPROACH

3.1 GENERAL

The main purpose of this study was to investigate the combined scour near groynes along the outer bank of a bend in alluvial channels and to determine how to combine bend scour and local scour, particularly in the case of bank erosion products having a significant influence on the shape of cross-sections in the unprotected situation. The study had to result in recommendation on how to assess the combined scour due to bend flow and groynes. If h_o is the water depth of a straight channel without groynes, h_b is the maximum depth of scour in outer bend and h_1 is the maximum scour depth at the head of groyne in a straight channel, then, with the construction of a groyne along the outer bank of a bend, the total depth of scour h_{tb} can probably be expressed as:

$$h_{tb} = f(h_o, h_1, h_b) \quad (3.1-1)$$

The exact formulation of this relation is not known. Experiments in a physical model with mobile bed and banks were carried out to clarify this relationship and thus to obtain a better prediction for the maximum scour to be expected around groynes in bends.

3.2 EXPERIMENTAL FACILITY USED

The tests were carried out in an experimental facility that was constructed in the De Voorst experimental station of Delft Hydraulics. The design of the facility was made on the basis of the design of the experiments (see Section 4.2). The characteristics of the facility used (see Fig. 3.2-1) are as follows:

- overall dimensions 9 m long and 6 m wide;
- the height of the walls is 0.60 m;
- a tailgate with the width of 3.0 m is installed at the lower side of facility to set the downstream water level;
- the facility is filled up with sand material having average particle size of $D_{50}=0.210$ mm;
- a closed system for sediment and water recirculation is used including a sand-trap, a water-storage, a sand pump with $Q = 7.75$ l/s, a clean water pump with a maximum discharge of 50 l/s and two separate pipe lines, see Fig. 3.2-1;
- a series of pipes with a diameter 45 mm and a length of 50 mm is used to dampen the turbulence of the flow downstream or where the flow is entering into the facility;
- the groynes were simulated with thin plates; the advantages of this schematization is (a) that they could be handled easily and (b) the local scour is independent of the scour depth, and
- several wooden rectangular plates with the dimensions 50 * 80 cm and 2 cm thickness were applied as groynes along channels (Fig. 3.2-2a). The main reasons for using these plate form groynes were (a) easy to handle and (b) the probability that any change in width of groyne as would be experienced for the (types b and c of Fig. 3.2-2 after erosion) would affect the scour depth around the head of groyne and consequently would jeopardize the comparison between the various experiments.

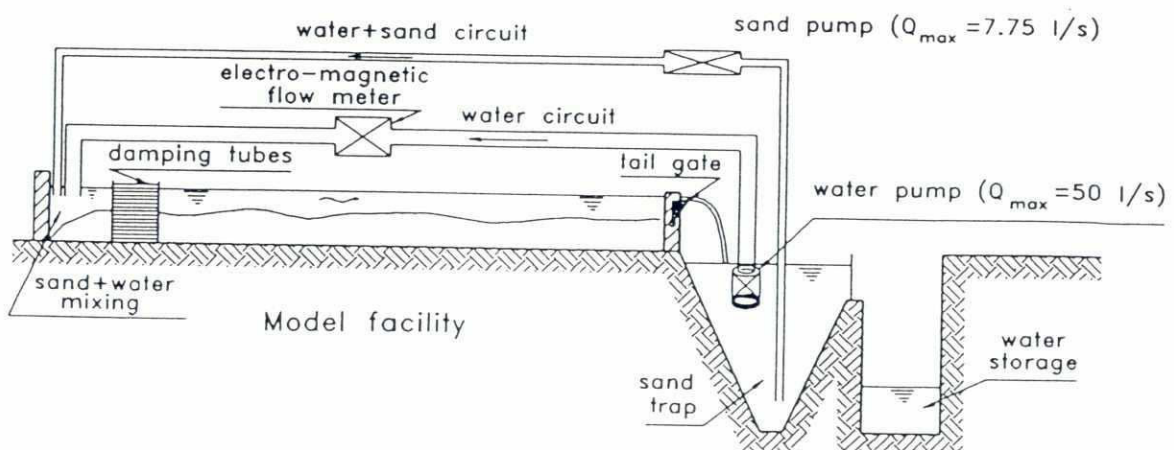


Fig. 3.2-1: Experimental facility used and water and sediment circuits

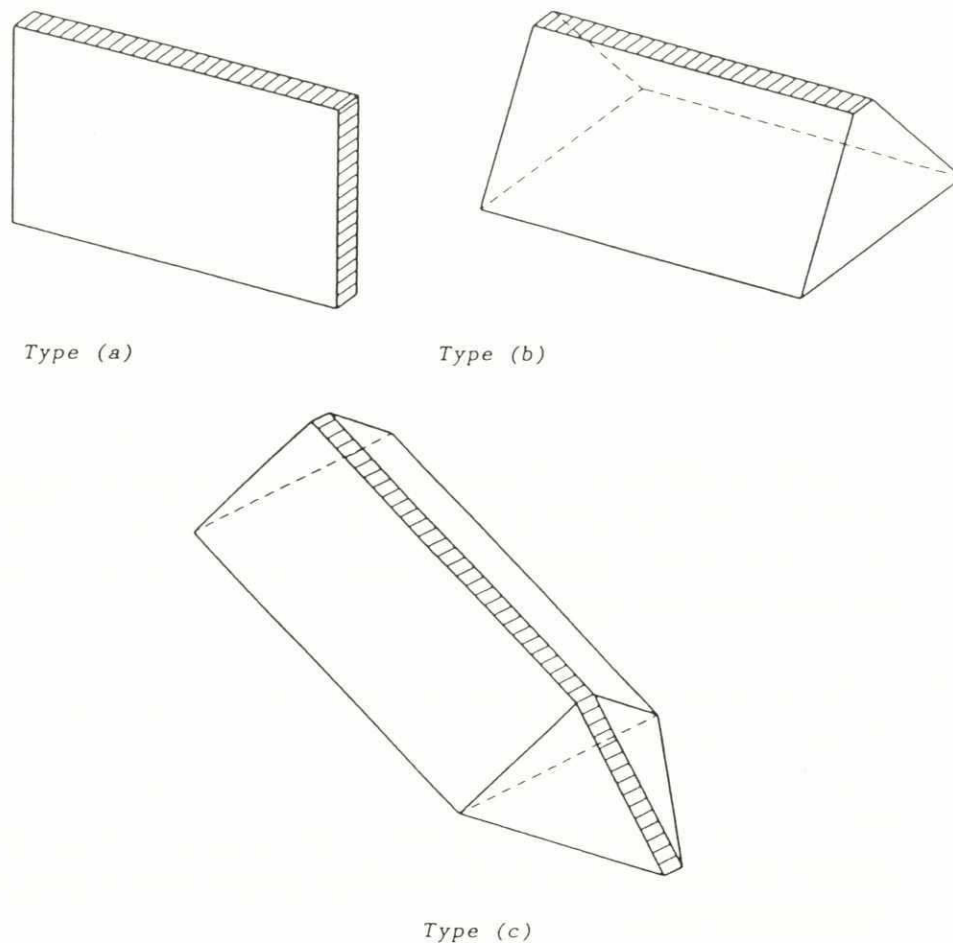


Fig. 3.2-2: Various types of groynes potentially useable in the experiments

The following measuring instruments were used:

- one electro-magnetic flow meter to measure the flow discharge;
- two Pitot tubes for measuring the water levels;
- one normal point gauge for measuring the bed levels, and
- one electro-magnetic flow velocity meter.

In addition vertical photographs were taken from a place about 7.5 m above the facility.

3.3 APPLIED PROCEDURES

In total three series of each five experiments were carried out. The five experiments are described hereafter as five subsequent stages of the investigation (see also Fig. 3.3-1):

Stage 1 : Running the model with a straight channel without any groynes. From this test the average water depth h_0 for given discharge Q , width B and slope i is obtained. During this experiment both banks were fixed.

Stage 2 : Running the model with a straight channel including groynes along one bank with the same Q , B and i . In this stage the aim is to determine the maximum depth of local scour h_l near the head of groynes and the increase in depth $\Delta h_l (= h_l - h_o)$ due to local scour. In this experiment only the bank without groynes was fixed.

Stage 3 : Running the model with a curved channel, without any groyne and with the same Q , B and i . The aim of stage 3 is to determine the maximum depth of outer bend scour h_b and the increase $\Delta h_b (= h_l - h_o)$ near the groynes. In this experiment both banks were fixed again.

Stage 4 : Running the model with a curved channel and groynes along the concave bank for the same conditions as the previous experiments to find the maximum depth of scour in front of groynes in the bend h_{lb} and the increase due to the combined presence of an outer bend and local scour $\Delta h_{lb} (= h_{lb} - h_o)$. In this experiment only the inner bend was fixed.

Stage 5 : In the last stage (only during the first two series of experiments) the same curved channel with erodible outer bank as in stage 4 was used. The groynes were placed in a retired position some 0.3 m from the outer bank. The initial conditions were the same bed topography during equilibrium conditions of the third stage. The purpose of this stage is to study the effects of bank erosion products on the bed topography of bends.

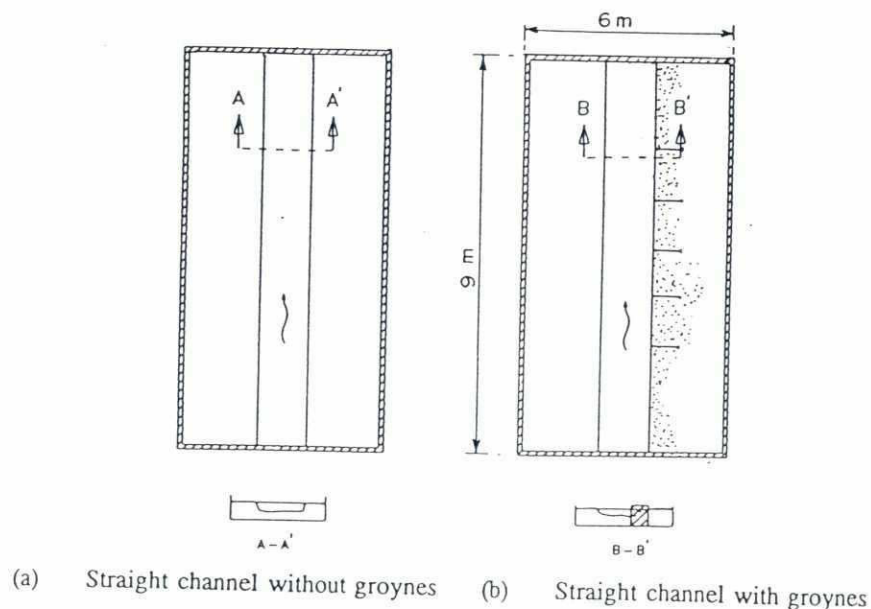


Fig. 3.3-1: Various stages of experiments

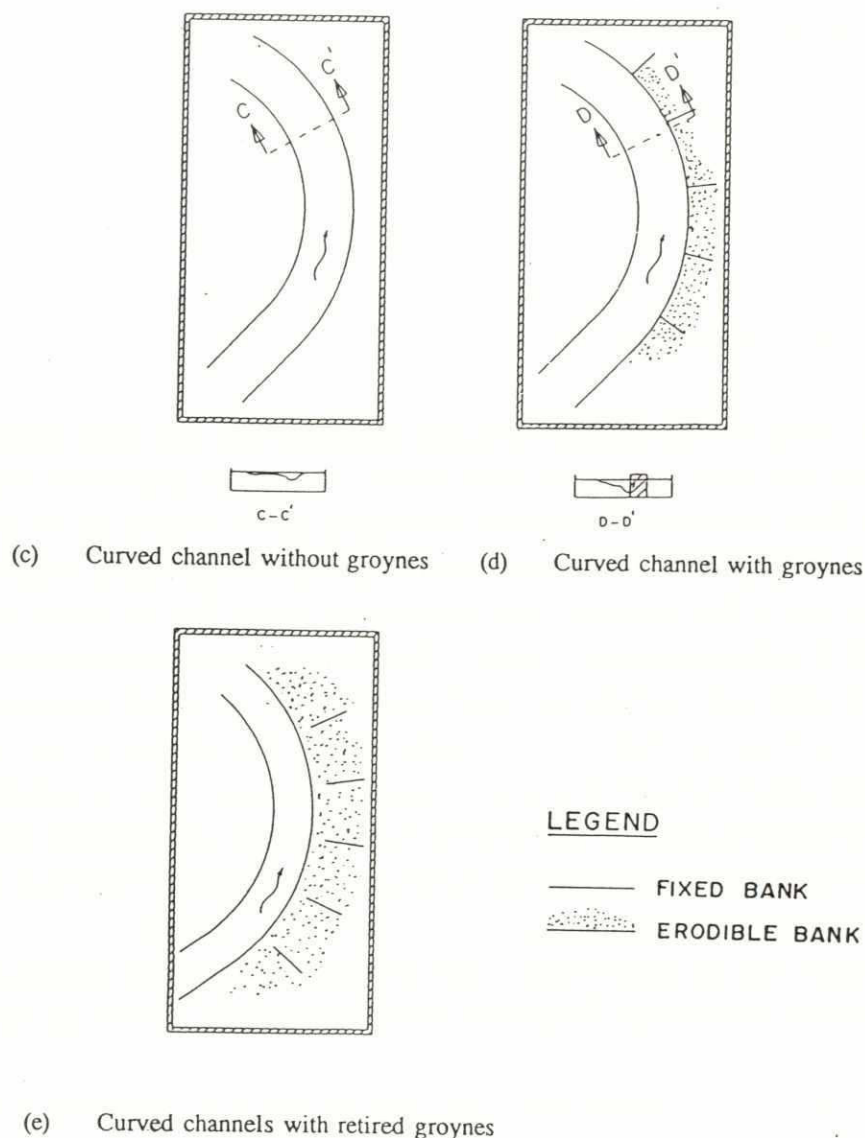


Fig. 3.3-1 (continued): Various stages of experiments

4 EXPERIMENTS CARRIED OUT AND RESULTS OBTAINED

4.1 INTRODUCTION

In this chapter a description is given of the experiments carried out and the results obtained are presented. Before doing this, however, the design of the experiments is discussed, mainly to explain the role of the independent and dependent variables. This issue is of importance because understanding of it is important to apprehend the differences between the initial characteristics of the experiments and their final characteristics. This is of relevance because during some experiments substantial differences did occur.

4.2 DESIGN OF EXPERIMENTS

In principle, the experiments are fully determined by the following parameters:

- bed material (density, size, gradation);
- width of channel;
- discharge, and
- slope of channel.

Hence, the hydraulic roughness (and thus the water depth) and the sediment transport become dependent variables.

In reality, however, the situation is slightly different. The first three parameters can indeed be induced and they remain the same (apart from some changes in width due to bank erosion). Also the slope of the channel can be pre-set via excavating the bed according to a certain slope. A problem arises due to the fact that the hydraulic roughness can only be estimated with a low accuracy (and develops in time). So the water depth for uniform conditions is only approximately known. Based on the best estimate the level of the tailgate is decided upon, but generally this level will be either too low or too high. Hence the flow conditions will deviate from uniform conditions, causing erosion or sedimentation near the tailgate and hence increased or decreased sediment transport near the tailgate (compared with uniform conditions). Because the sediment is recirculated, the sediment transport takes over the role of independent variable from the slope of the channel. Therefore, the results of the tests become to some extent unpredictable. During the experiments this was experienced as the main drawback for the procedure used, because every time the ultimate slope differed too much from the target slope. This made mutual comparison of the tests more difficult.

Nevertheless the initial design was done using the four parameters listed in the first part of this section as independent parameters, so including the channel slopes, and during the experiments it was attempted to approach these conditions as much as possible.

Because the results of the experiments are meant to be used in conjunction with the design of river training works along the Jamuna river, an attempt was made to simulate the conditions in this river as much as possible during the experiments. Because it was decided in an early scope to use fairly uniform sand with a diameter of about 0.2 mm as bed material, this meant that the scaling procedure was essentially the same as used for the movable bed model used with the Jamuna Bridge Project, Phase II (see Klaassen, 1990b) and the considerations as discussed in the following paragraphs are of relevance.

Reproduction of morphological processes into scale models considerably enlarges the complexity of scaling. Both water-movement and sediment transport have to be reproduced simultaneously. The growing insight into sediment transport in alluvial channels and morphological phenomena in rivers has given a sound basis for this type of model. The interaction of sediment and water movement can, however, still lead to large errors if no

check on the scaling is carried out, so calibration is essential in mobile bed models. Hence prototype data are needed.

To reproduce a morphological process correctly in the scale model, it is necessary to consider various scale conditions. Here the scale of a parameter (X) is defined as the ratio of its value in the prototype and in the model, such as:

$$n_x = \frac{X_p}{X_m} \quad (4.2-1)$$

First of all, the Froude condition has to be fulfilled for the free surface flow present. Secondly, the roughness condition has to be considered. For river studies, curvature of stream lines has to be reproduced properly, otherwise a bed configuration as a result of an interaction between water movement and sediment movement will be wrong. It can be remarked that, if Froude condition and roughness condition are fulfilled, then automatically the slope is reproduced correctly.

The **Froude condition** is fulfilled if:

$$n_v = (n_h)^{1/2} \quad (4.2-2)$$

n_v and n_h being the scales for velocities and depths respectively.

But for mobile bed models the flow velocities according to Froude's law would give insufficient movement of the bed material (if sand is applied), so deviation of the Froude condition is required and the effects of this deviation should be considered.

The **roughness condition** is fulfilled if:

$$n_c^2 = \frac{n_L}{n_h} = r = \text{distortion} \quad (4.2-3)$$

n_c and n_L being the scale for widths and lengths.

For mobile bed models of relatively wide rivers like the Jamuna, the roughness condition is very important.

The main reason for the current study is to determine the maximum depth of scour in three series of experiments for local scour, bend scour and the combination of bend and local scour. Then, by comparing the results, it is possible to find the influence of bend flow on the local scour around the constructed groynes along the outer bank of a curved channel. For a correct reproduction of the bend scour (and hence the overall bed topography) the conditions to fulfill are a.o.: (i) Froude condition, (ii) Shields condition, yielding the "ideal velocity scale" (see Jansen, 1979) and (iii) undistorted model. For local scour important conditions are: (a) Froude condition, (b) undisturbed model, and (c) velocities in the model

above the critical velocity (see Jamuna Bridge Phase II Study). A further analysis shows that for sand as bed material, full reproduction of bend scour and local scour is not possible. Hence scale effects will always be present.

Since for the present series of experiments the **comparison** of the results is dominant, there will be no objection against using a model which is subject to scale effects, although it makes the direct translation of the results to prototype conditions not necessary. The main result to be expected from the investigation is how to **combine** local scour and bend scour, and insight into this may be obtained. It is necessary, however, to conduct this series of experiments under the same conditions; for instance by using the same grain size and density of sediment, the same Chézy coefficient and the same flow velocity at the cross-section of the scour hole. Also, it is important to reach the equilibrium condition of channel so as to determine the maximum scour depth.

For design of experiments and experimental facility, various aspects should be considered and applied. In Fig. 4.2-1 these aspect and the procedure of design for the experiments and the experimental facility are shown.

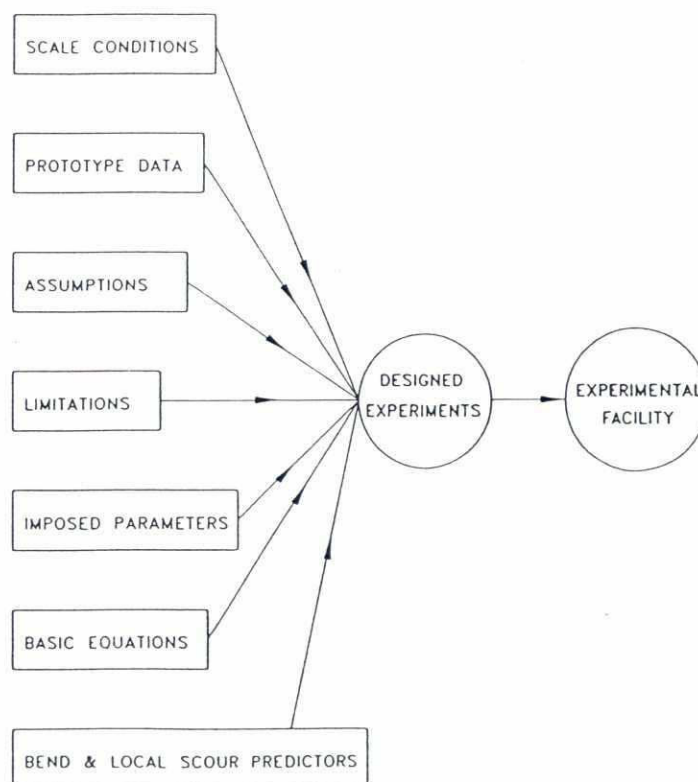


Fig. 4.2-1: Schematic indication of design procedure for experiments and experimental facility

For information on the prototype conditions reference is made to Table 4.2-1.

Total average discharge (Q_T)	50,000 m^3/s
Average discharge in the main channel (Q)	20,000 m^3/s
Width of main channel for the average discharge (B)	1500 m
Average depth of water in the main channel (h)	8 m
Chézy coefficient (C)	70 $\text{m}^{1/2}/\text{s}$
Slope (i)	$7 \cdot 10^{-5}$ (-)
Median size of sand as bed material (D_{50})	200 μm
Shields parameter (θ)	1.7 (-)
Average velocity ($\bar{U}$)	1.67 m/s
Average total bed material transport (s_t)	$1.8 \cdot 10^{-3}$ $\text{m}^3/\text{m/s}$
Number of main channel per reach	2-3

Table 4.2-1: The characteristics of the Jamuna river

To fulfill the objective of this study, also for the budget constrain and for some simplifications, the following assumed conditions and limitations for the design were accepted.

Assumed conditions

- (i) Channel flow is shallow friction controlled.
- (ii) Channel width is initially constant.
- (iii) Discharge is constant during each test (time independent).
- (iv) The flow field is mainly governed by the bed topography, the roughness condition and spiral motion (in bend).
- (v) The sediment transport scale is constant in space for correct reproduction of the bed topography.
- (vi) The bed is erodible and the banks are non-erodible in the stages 1 and 3 but one bank will be erodible in the stages 2, 4 and 5.

Limitations

- (i) Froude number can be accepted in the range of small to moderate (up to 0.65).
- (ii) The facility to be used would have approximate dimensions of 10 m*5 m.
- (iii) The depth of scour should not be more than 30 cm because of the limitation of sand layer thickness in the existing facility.
- (iv) Sand with $D_{50} = 0.210$ mm will be used as bed material for the following reasons: (a) the limited availability of light-weight materials in the laboratory, (b) to avoid difficult handling procedures and (c) the high cost of purchasing light materials.

The design of the tests was carried out considering the following points:

1. The data in Table 4.2-1 are the main information of the prototype for the design of the tests.
2. The assumed conditions and limitations as specified above were considered during the design of the experiments.
3. The imposed parameters are:
 - using sand with $D_{50} = 0.210$ mm as sediment material;
 - the initially selected discharges were 10, 14 and 20 l/s, and
 - the initially selected slopes were 0.003, 0.005 and 0.007.
4. These following laws and equations were used to determine the scales and dimensions of the model:
 - the continuity law of flow $Q = A * U$ (4.2-4)
 - the regime equation $n_h^- = n_Q^{1/3}$ (4.2-5)
 - the Chézy equation $U = C \sqrt{hi}$ (4.2-6)
 - the Froude number formula $Fr = \frac{V}{\sqrt{g h}}$ (4.2-7)
5. The predictors of Ahmad, Lacey, Gill and Garde et al. are used to determine the maximum depth of local scour in front of groynes in straight and curved channels.
6. The methods of Zimmermann and Kennedy (1978), Odgaard (1984) and Struiksma (1988) are used to determine the maximum bend scour in the curved channel.
7. Due to the limitation of the existing facility, and considering the prototype data and the curvature needed for the experiments, the radius of curvature for the curved channel was selected 3.5 m in flume.

More details on the procedure for designing of the experiments can be found in Mesbahi (1992). Here only the ultimate results are presented in Table 4.2-2. It concerns the selected imposed variables for each possible experiment, together with the predicted bend scour and local scour depth. In the end from these 9 (nine) possible series of experiments 3 (three) were used for the present study.

Case no.	D_{50} (mm)	Δ	i	Q (m ³ /s)	h (m)	C (m ⁴ /s)	v (m/s)	B (m)	F_r	θ	R_c (m)
1	0.21	1.65	0.001	0.020	0.080	24	0.215	1.16	0.243	0.23	3.5
2	0.21	1.65	0.001	0.030	0.092	24	0.230	1.43	0.242	0.27	3.5
3	0.21	1.65	0.001	0.040	0.100	24	0.240	1.67	0.242	0.29	3.5
4	0.21	1.65	0.003	0.020	0.080	24	0.370	0.68	0.418	0.69	3.5
5	0.21	1.65	0.003	0.030	0.092	24	0.400	0.82	0.421	0.80	3.5
6	0.21	1.65	0.003	0.040	0.100	24	0.416	0.96	0.420	0.87	3.5
7	0.21	1.65	0.005	0.020	0.080	24	0.480	0.52	0.540	1.15	3.5
8	0.21	1.65	0.005	0.030	0.092	24	0.515	0.63	0.542	1.33	3.5
9	0.21	1.65	0.005	0.040	0.100	24	0.537	0.74	0.542	1.44	3.5

(a) Characteristics of the experiments

Table 4.2-2: Characteristics of possible experiments and predictions of bend scour and local scour

case no.	Bend Scour Depth (h_b)			Local Scour Depth (h_l)			
	Struiksma	Odgaard	Zimmerman	moderate bend		straight channel	
	h_b	d_o	d_i	Lacey	Ahmad	Garde	Gill
1	0.124	0.123	0.110	0.206	0.196	0.134	0.175
2	0.184	0.157	0.140	0.235	0.210	0.153	0.194
3	0.224	0.186	0.167	0.260	0.231	0.167	0.206
4	0.140	0.123	0.132	0.206	0.266	0.216	0.173
5	0.192	0.157	0.176	0.235	0.308	0.250	0.192
6	0.234	0.186	0.216	0.260	0.334	0.271	0.204
7	0.146	0.123	0.147	0.206	0.317	0.270	0.173
8	0.192	0.156	0.199	0.235	0.367	0.312	0.192
9	0.251	0.185	0.248	0.260	0.399	0.339	0.204

(b) Predictions of bend scour and local scour

Table 4.2-2 (continued): Characteristics of possible experiments and predictions of bend scour and local scour

It is remarked that predicted scour depths are up to 0.4 m below the initial bed level. This is the reason that the existing tilting facility was not appropriate for the present study. It was decided to build a new facility for this investigation.

4.3 DESCRIPTION OF THE EXPERIMENTS

Further on the approach described in Section 1.3 and in more detail in Section 3.3 it was decided to first conduct a complete series of tests including the five stages explained in Section 3.3 with the same imposed parameters, starting with a straight channel (stage 1) and ending with an experiment with mobile banks (stage 5). The idea was to gain practical experience with the different aspects of the tests, such as duration of each test, the method of imposing the same conditions for various stages, actual problems of the experiments and also the preliminary results of the tests. After that, the other tests were conducted with the different discharges, initial slopes and width of the channel.

In the remaining part of this section a complete series of tests will be described and afterward the total results of all tests will be described in Section 4.4. It should be mentioned that the number of the cross-sections is related to the distance of each one from the beginning of the channel on the centreline. It is stressed that the final conditions of each experiment deviated to some extent from the initial characteristics because of the impossibility to fully impose the test conditions (conform the previous section)

In the first series five tests were conducted in five stages, each of which will be described hereafter with the following codes:

- SCWG = Straight Channel Without Groynes (Stage 1)
- SCIG = Straight Channel Including Groynes (Stage 2)
- CCWG = Curved Channel Without Groynes (Stage 3)
- CCIG = Curved Channel Including Groynes (Stage 4)
- CCWIG = Curved Channel Without and Including Groynes (Stage 5)

The number added to this code indicates the series number.

1. Test SCWG1 (straight channel without groyne of the series 1)

This test was conducted with the fixed banks (of wooden plates 4 mm thick) which had the side slope 3:1. Initially a test with a straight channel with erodible banks was carried out, but due to continuous widening without equilibrium condition, this appeared not to be feasible.

The initial characteristics of the test SCWG1 are as follows:

Q	= 0.021 m ³ /s
I _(i)	= 0.001
B	= 0.80 m
D ₅₀	= 0.210 mm
start time	= 16:30 of 13-3-92
end time	= 10:00 of 16-3-92
effective time	= 73 hours

During the test, various cross-sections of the channel were sounded using a metal bridge and a simple point gauge to control the bed changes. The water level was measured using two static Pitot tubes to determine the water slope. Both measurements were used to check the equilibrium condition of the channel being a uniform flow with parallel bed and water surface. The equilibrium of this test is shown in Fig. 4.3-1 for the last three soundings. In the equilibrium conditions,

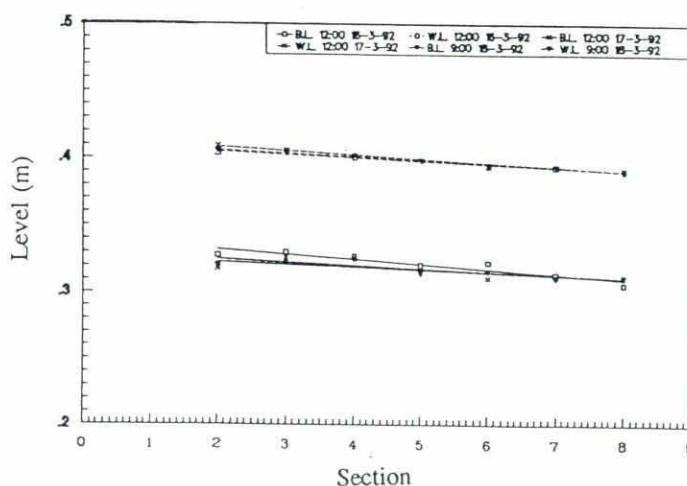


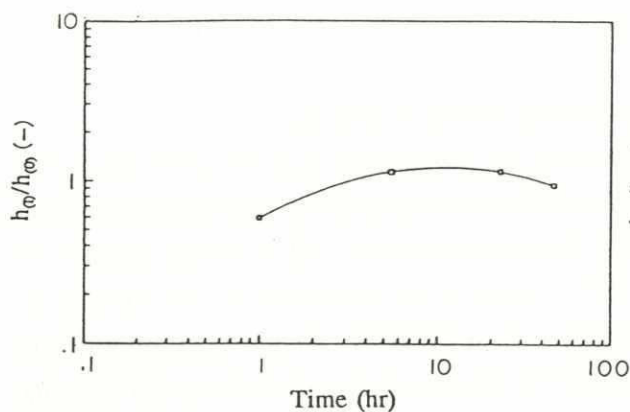
Fig. 4.3-1: Controlling the equilibrium of test SCWG1

the flow velocity was measured using an electro-magnetic velocity meter at three points of each cross-section in an average of 100 seconds. Also several series of sediment transport samples were obtained at the downstream and along the whole width of the tail gate, to determine the sediment transport of the channel.

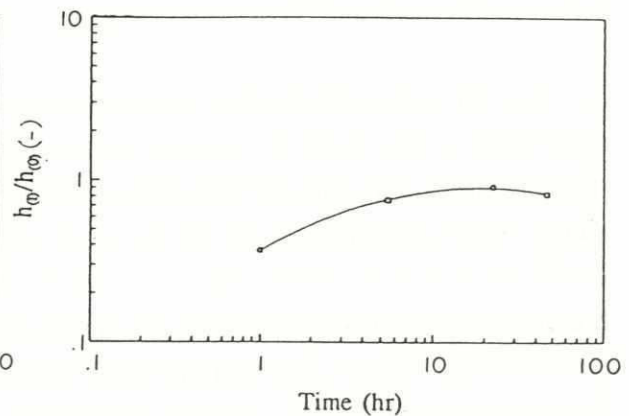
2. Test SCIG1 (straight channel including groyne of the series 1)

Test SCIG1 was carried out with the following conditions:

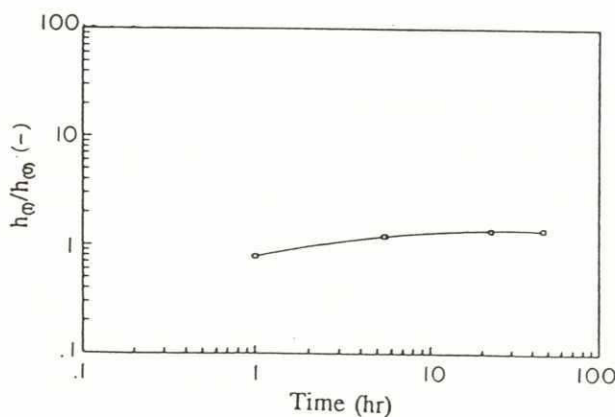
- the bed topography and the water slope were kept the same as the equilibrium condition of test SCWG1;
- the wooden plates on the right bank of the channel were removed;
- the wooden rectangular 12 mm thick plates were applied as the groynes along the erodible bank;
- the width of the channel in front of the groynes was kept 0.8 m (the same width as in test SCWG1) to neglect the effect of constriction on the local scour depth, and
- the discharge was also 0.021 m³/s.



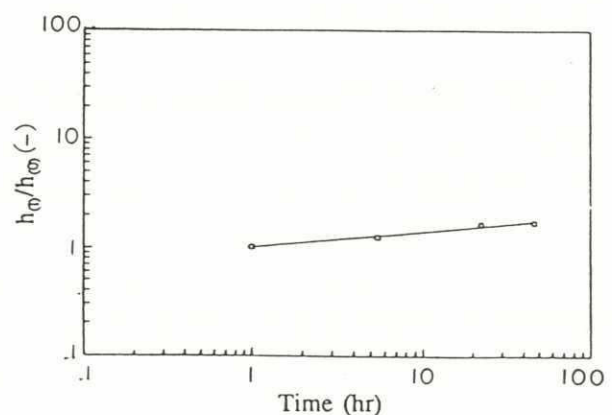
a. Section 4



b. Section 5



c. Section 6



d. Section 7

Fig. 4.3-2: Controlling the equilibrium of the test SCIG 1.2

As an additional investigation about the effect of spacing between the groynes on the maximum local scour depth around the spurs, this test was carried out in three parts (called SCIG1.1, SCIG1.2 and SCIG1.3) with spacings 0.5, 1.0 and 2.0 m, respectively, see Photo 1 and 2.

To control the equilibrium condition right from the beginning of the test, every few hours interval the cross-sections of the channel in front of the groynes were sounded. By determining the maximum local scour depth a logarithmic graph of h_i/h_o and t (time) was prepared. The horizontal reach or downward direction of the curved line in the graph, can denote the equilibrium condition, see Fig. 4.3-2. Of course it should be mentioned that if the experiments were continued, these graphs would have fluctuated around the equilibrium condition of relative scour depth (h_i/h_o).

In the equilibrium condition, the final soundings and velocity measurement in front of groynes and between the groynes (in the next series) were done. After the test had stopped the scour holes around the groynes were sounded. The running time of the three parts of this test was 49, 46.5 and 39.5 hours, respectively.

3. Test CCWG1 (Curved channel without Groyne of the series 1)

After transferring from the straight channel situation to the curved channel with the radius of curvature 3.5 m, the test was carried out with the fixed banks and the same discharge, bed slope, width and position of the tail gate as in the previous tests, ($Q = 0.021 \text{ m}^3/\text{s}$, $i = 0.001$, $B = 0.8 \text{ m}$). The facility was run for 57.5 hours so as to obtain the equilibrium of the channel which was controlled in the same way of test SCWG1. At the beginning of the channel there was a 1.6 m straight reach and the 6.5 m long second part was curved. During the equilibrium condition, the final bed sounding, water level and velocity measurements in each cross-section (with 1 m distance) and sediment transport sampling were performed. In particular in this test the transverse bed slope, the outer bend scour depth, the pool and point bar of the bed configuration were considered. Photo 3 and 4 show the situation of the channel during and after the test, respectively.

4. Test CCIG1 (Curved Channel Including Groyne of series 1)

This test began as the fourth stage of the first series of tests removing the fixed plates from the outer bank of the channel and installing six groynes (at 1.1 m distances) along the outer bank without any constriction of width ($B = 0.8 \text{ m}$ in front of the groynes). The bed topography, discharge and the position of the tail gate were kept the same as in the equilibrium condition of test CCWG1. Very soon, the bed around the head of the groynes began to erode, and after some time a large and two small eddies were created between each two groynes. It was also clear that there were strong vortices in front of the groynes. The equilibrium condition of the test was controlled in the same way as in test SCIG1 (Fig.4.3-3). All final measurements were done after ascertaining the equilibrium conditions.

At the end of the test (run time of the test was 46 hours) the scour holes around the groynes were sounded. Photo 5, 6 and 7 show the condition of the test during and after running the model and the scour holes around the groynes.

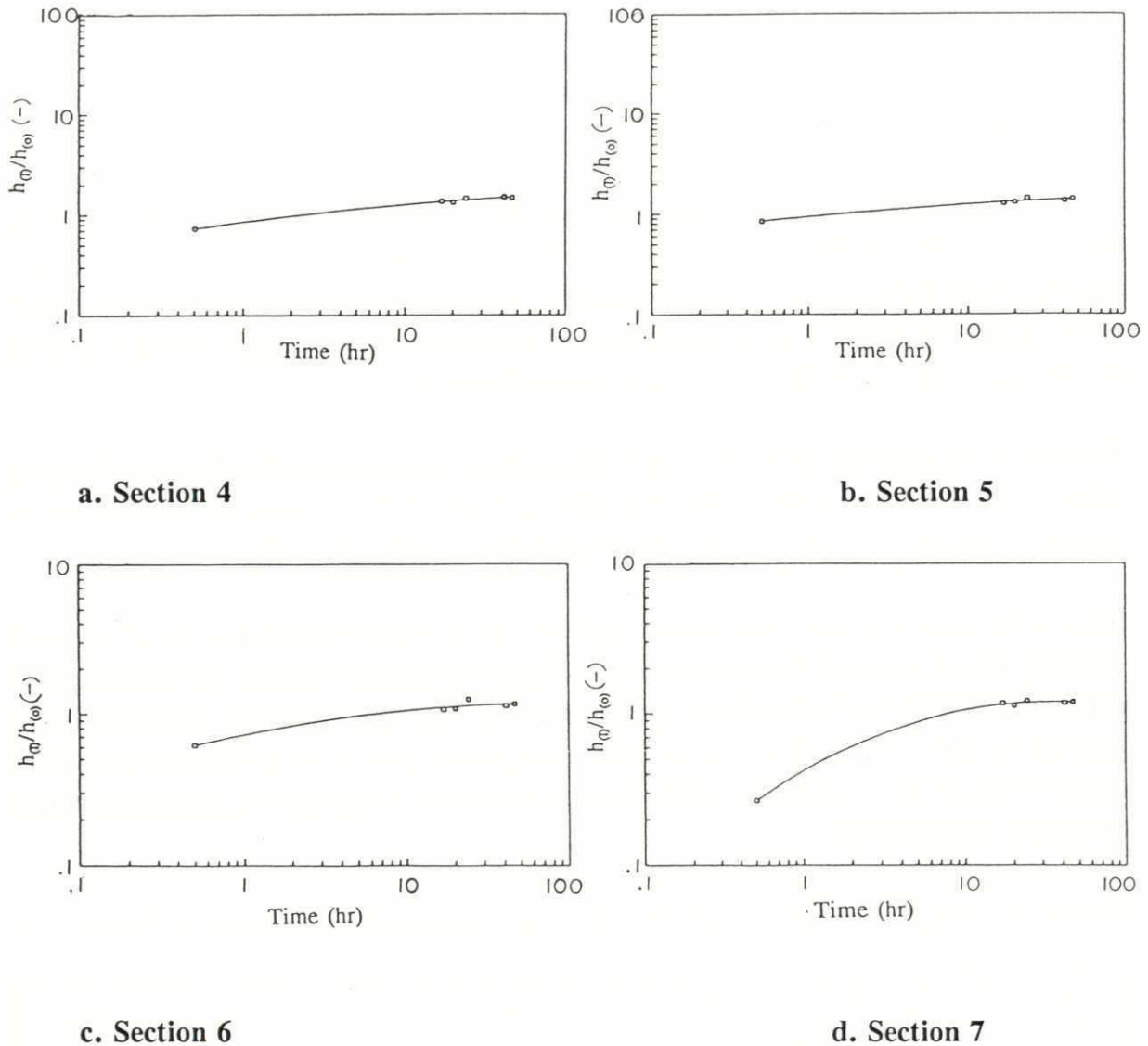


Fig. 4.3-3: Controlling the equilibrium of the test CCIG1

5. Test CCWIG1 (Curved Channel without and Including Groyne of series 1)

The main purpose of the last test of each series of experiments was to study the effect of bank erosion products on the bend scour depth or transverse bed slope. In other words, the change of the bed level in two different cases, notably (a) with protected outer bank or $\partial h/\partial t = 0$, (b) with unprotected outer bank ($\partial h/\partial t \neq 0$) to find a correct criterion for design of groynes along outer bend were studied. Therefore, it was decided to conduct this test in two parts.

In the first part, the curved channel with fixed banks and without groynes was runned to find the maximum bend scour depth along the outer bend in the equilibrium condition. In order to reduce the time to attain equilibrium condition, the channel bed topography was constructed in the same condition as with the equilibrium of test CCWG1.

The second part had begun just after condition to the equilibrium of the first part with the removal of the fixed plates of the outer bank. In this part, the groynes were installed 0.30 m away from the outer bank along the channel to limit the bank erosion and so the widening of the channel. Also the rate of bank erosion was determined. So during the test, the bed level soundings were performed over short periods (30 minutes) and also the photographs were taken from the top over the facility. The bank erosion started immediately after removing the plates and continued rapidly. The bed of the channel rose up and after 3 hours, due to the effect of the groynes, the bed around the groynes began to lower. During the test mass failures of the outer bank were considerable.

The total run time of test CCWIG1 was 29 hours and it was stopped after observing the great effect of the local scour on the bed topography. Photo 8 to 11 show the position of the channel in the first and second part of test CCWIG1 and after the test.

It has to be mentioned that in this type of experiments, due to the position of the groynes and widening of the channel, the flow direction changed strikingly towards the middle of the outer bank and that it had a direct contact with the bank and especially upstream of the groynes located at that position so the scour hole pattern around the groynes differed from those in tests CCIG. Usually the depth of scour was more at upstream of groynes than next to them in tests CCWIG.

Finally, the initial characteristics of the various test conducted can be seen in Table 4.3-1.

Test Number	Test Code	Q(w) (m ³ /s)	i(i) (-)	B(i) (m)	L (m)	R(c) (m)	D ₅₀ (mm)
1	SCWG1	.021	.003	.80	7.00	-	.21
2	SCIG1.2	.021	.00256	.80	7.00	-	.21
3	CCWG1	.021	.003	.80	8.10	3.5	.21
4	CCIG1	.021	.0024	.80	8.10	3.5	.21
5	SCWG2	.030	.003	.96	7.00	-	.21
6	SCIG2	.030	.00322	.96	7.00	-	.21
7	CCWG2	.030	.003	.96	8.10	3.5	.21
8	CCIG2	.030	.00345	.96	8.10	3.5	.21
9	SCWG3	.030	.001	.96	7.00	-	.21
10	SCIG3	.030	.0014	.96	7.00	-	.21
11	CCWG3	.030	.001	.96	8.10	3.5	.21
12	CCIG3	.030	.00305	.96	8.10	3.5	.21

Table 4.3-1: Initial characteristics of the tests **

** Tests SCIG1.1, SCIG1.3, CCWG1, CCWIG1, CCWIG2 and CCWIG3 are not included in the table.

4.4 RESULTS OF THE TESTS

Based on what is explained in Section 4.3, the information obtained from the tests, include:

- bed level of cross-section (z_b)
- water level (z_w)
- velocity (3 points in each cross-section) (v)
- discharge (Q_w)
- sediment transport samples (for the tests without groyne)
- slope of water surface

Processing of the obtained data is necessary for interpretation of the results. They are needed to calculate the average parameters of each test, such as:

- average water depth ($\bar{h}$)
- average velocity ($\bar{v}$)
- average Froude number ($\bar{Fr}$)
- average Chézy coefficient ($\bar{C}$)
- average Shields parameter ($\bar{\theta}$)

and also to determine:

- average water depth in the tests (SCWG) (h_o)
- maximum water depth in front of groynes of tests (SCIG) (h_i)
- maximum water depth in outer bend of tests (CCWG) (h_b)
- maximum water depth in front of groynes of tests (CCIG) (h_{lb})
- discharge of sediment transport of tests (SCWG & CCWG) (q_s)

The results of the various tests obtained after having processed the data are shown in Table 4.4-1.

Test number	Test code	Run. time (hr)	i(f) (-)	$\bar{h}$ (m)	h(max) in the sections:					$\bar{v}$ (m/s)	$\bar{Fr}$ (-)	$\bar{\theta}$ (-)	$\bar{C}$ ($m^{0.5}/s$)	q(s)*10 ⁻⁶ (m ² /s)
					3	4	5	6	7					
1	SCWG1	73	.0025	.08	.077	.077	.084	.079	.084	.347	.392	.58	23.34	5
2	SCIG1.2	46.5	.00276	.108	.095	.164	.18	.181	.171	.307	.351	.661	24.18	-
3	CCWG1	57.5	.0024	.089	.118	.177	.179	.18	.099	.347	.371	.616	20.9	2.4
4	CCWG1	46	.00267	.128	-	.257	.27	.25	.244	.271	.268	.844	15.09	-
5	SCWG2	49.5	.00322	.064	.066	.066	.066	.066	.063	.544	.697	.599	33.74	7.65
6	SCIG2	95	.00376	.055	.125	.121	.119	.131	.156	.572	.809	.594	46.79	-
7	CCWG2	113	.00345	.076	.093	.163	.225	.136	.129	.496	.583	.793	27.81	23.24
8	CCIG2	48.25	.00367	.091	.141	.246	.305	.27	.268	.415	.454	.967	20.7	-
9	SCWG3	94	.0014	.096	.088	.091	.098	.102	.097	.359	.371	.386	28.43	2.9
10	SCIG3	115	.00236	.071	.1	.114	.127	.163	.177	.415	.514	.484	39.13	-
11	CCWG3	94	.00305	.094	.108	.158	.144	.199	.225	.44	.466	.827	21.37	22.12
12	CCIG3	23	.00463	.082	.115	.229	.253	.272	.258	.498	.6	1.09	24.06	-

Table 4.4-1: Characteristics of the tests before correction

Since there were some data missing in tests SCIG1.2 and CCIG1, it was decided to modify them. The problem was that the bed and water level and velocity measurements were not performed between the groynes. Therefore, the results of processing the data for these two tests presented in Table 4.4-1 could not be confirmed. To modify these data a method based on the Chézy- and continuity equations was used, and is explained in detail in Mesbahi (1992).

Another modification of the data was done in tests CCWG2 and CCWG3. Because there were some fluctuation of bed and water level of the channel during the equilibrium condition, it was decided to use the average parameters of the last three measurements only. For more details also reference is made to Mesbahi (1992). The partly modified data of the different tests after making these corrections are shown in Table 4.4-2. Here it concerns the average conditions. For six tests the sediment transport per metre width of the channels in the tests without groynes were determined by sampling of the water falling over the tailgate.

Test number	Test code	Run. tim (hr)	i(f) (-)	$\bar{h}$ (m)	h(max) in the sections:					V(ave) (m/s)	Fr(ave) (-)	O(ave) (-)	C(ave) (m.5/s)	q(s)*10 ⁶ (m ² /s)
					3	4	5	6	7					
1	SCWG1	73	.0025	.08	.077	.077	.084	.079	.084	.347	.392	.58	23.34	5
2	SCIG1.2	46.5	.00276	.083	.095	.164	.18	.181	.171	.307	.351	.661	24.18	-
3	CCWG1	57.5	.0024	.089	.118	.177	.179	.18	.099	.347	.371	.616	20.9	2.4
4	CCIG1	46	.00267	.11	-	.257	.27	.25	.244	.271	.268	.844	15.09	-
5	SCWG2	49.5	.00322	.064	.066	.066	.066	.066	.063	.544	.697	.599	33.74	7.65
6	SCIG2	95	.00376	.055	.125	.121	.119	.131	.156	.572	.809	.594	46.79	-
7	CCWG2	113	.00345	.076	.093	.163	.225	.136	.129	.496	.583	.793	27.81	23.24
8	CCIG2	48.25	.00367	.091	.141	.246	.305	.27	.268	.415	.454	.967	20.7	-
9	SCWG3	94	.0014	.096	.088	.091	.098	.102	.097	.359	.371	.386	28.43	2.9
10	SCIG3	115	.00236	.071	.1	.114	.127	.163	.177	.415	.514	.484	39.13	-
11	CCWG3	94	.00305	.094	.108	.158	.144	.199	.225	.44	.466	.827	21.37	22.12
12	CCIG3	23	.00463	.082	.115	.229	.253	.272	.258	.498	.6	1.09	24.06	-

Table 4.4-2: Final modified characteristics of the tests

5 INTERPRETATION OF RESULTS

5.1 INTRODUCTION

In this chapter the Consultant will try by analyzing the data described in Section 4.4, to interpret them to find a relation between original water depth, local scour depth and bend scour with the combined scour in front of groynes in bends.

As explained in Section 3.3, it is necessary to have the same conditions of flow to compare the results of the various stages of each series of the tests. In spite of the modification of the data, the flow conditions in the third series are not acceptable, due to the variations in water slope and Shields parameter. Hence it was decided not to use the results of the third series and to elaborate only the data of the first and second series. Based on these acceptable data,

which were not enough but the only one available, a comparison could be made that yielded conclusions regarding how to determine the combined effect of bend and local scour for groyne in outer bends. Furthermore, the results of the test were compared with several predictors of bend scour and local scour around the groynes in the straight and curved channels. Finally, the effects of bank erosion on the bend scour are discussed in this chapter.

Tables 5.1-1a and 5.1-1b show the data of the first and second series for the purpose of comparison.

a. First series:							
Test: Section	SCWG1 h(o)	SCIG1.2 h(l)	CCWG1 h(b)	CCIG1 h(lb)	$\Delta h(l)$	$\Delta h(b)$	$\Delta h(lb)$
3	.077	.095	.118	-	.018	.041	-
4	.077	.164	.177	.257	.087	.100	.180
5	.084	.180	.179	.270	.096	.095	.186
6	.079	.181	.180	.250	.102	.101	.171
7	.084	.171	.099	.244	.087	.015	.160
b. Second series:							
Test: Section	SCWG2 h(o)	SCIG2 h(l)	CCWG2 h(b)	CCIG2 h(lb)	$\Delta h(l)$	$\Delta h(b)$	$\Delta h(lb)$
3	.066	.125	.093	.141	.059	.027	.075
4	.066	.121	.163	.246	.055	.097	.180
5	.066	.119	.225	.305	.053	.159	.239
6	.066	.131	.136	.270	.065	.070	.204
7	.063	.156	.129	.268	.093	.066	.205

Table 5.1-1: Various water depths of the first and second series of tests (values in metres)

The following nomenclature is used in Table 5.1-1:

$$\Delta h(l) = h(l) - h(o)$$

$$\Delta h(b) = h(b) - h(o)$$

$$\Delta h(lb) = h(lb) - h(o)$$

5.2 BEND SCOUR

During the experiments described here half of the experiments were carried out in a curved flume. Unfortunately, however, the flow conditions in experiments that should have been completely comparable, did differ to some extent. This holds especially for the slope of the water level (and hence the bed during uniform equilibrium conditions) and for the sediment transport. Because still the results of these experiments have to be used for comparison, an attempt was made to investigate the effect of the differences in water level slope on the phenomena in the bends, and to decide whether a correction was possible and needed.

For such an analysis of the conditions in the bends, it is necessary to make a distinction between the axisymmetric conditions and the conditions in the beginning of the bends. It will

be demonstrated here that in none of the experiments axisymmetric conditions were present, because the length of the "beginning of the bend" (as "defined" in Section 2.2.5) was much larger than the bend itself. The conditions in the beginning of a bend are also affected by the axisymmetric conditions, even if the bend is not long enough. If the bend is too short as in the present case, it is important to try to differentiate the superimposed overshoot (point bar) phenomena (Section 2.2.5) from the axisymmetric "base tone" (Section 2.2.4). This is schematically indicated in Fig. 5.2-1.

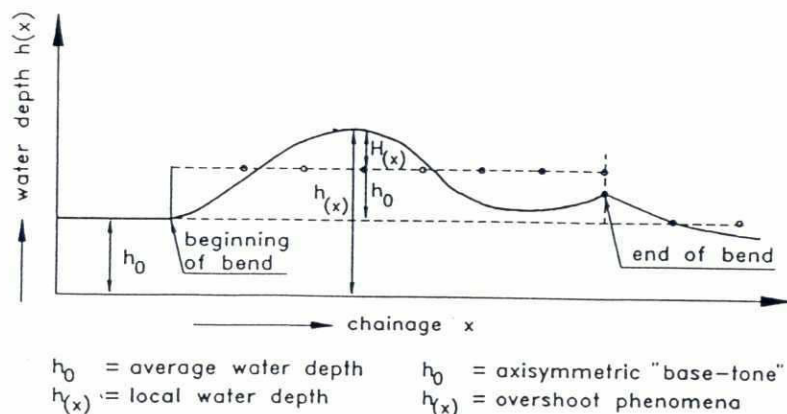


Fig. 5.2-1: Schematized differentiation of the superimposed overshoot phenomenon from the axisymmetric "base tone" in short bend

Conform what is stated in Section 2.2.6 the bed level in the outer bend is determined by:

- axisymmetric conditions (see Section 2.2.5);
- the values of λ_w , λ_s and the power b (see Section 2.2.6), and
- upstream conditions.

It is assumed that the upstream conditions were the same in all experiments, notably a uniform distribution of the flow and the sediment transport over the width (see also Section 6.2 for a discussion on this aspect). Hence the last aspect can be excluded from the considerations. This leaves the first two as important elements.

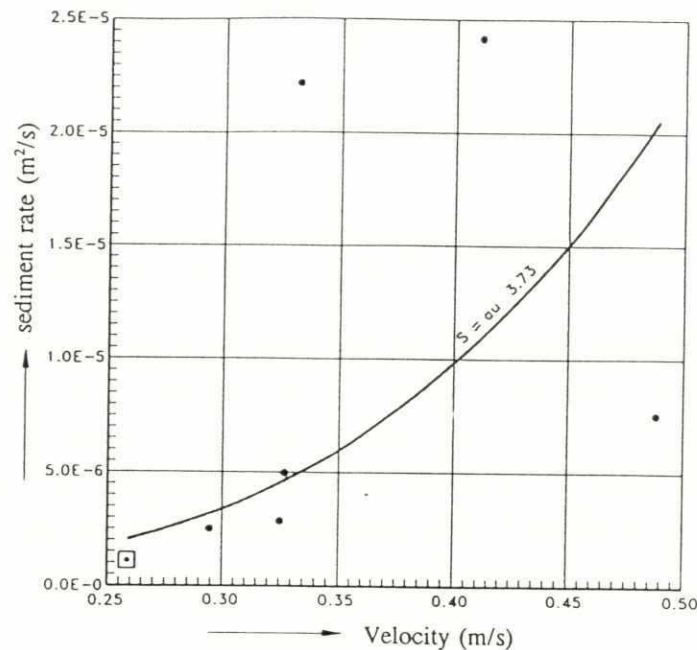
It is of interest to compare the various experiments as far as their adaptation towards axisymmetric conditions is concerned. This is done on the basis of the measured parameters and the derived parameters. These derived parameters are ultimately the periodicity, determined by the parameter L_p and the dampening, characterized by the dampening length L_d . In addition the value of the parameter b should be known.

An attempt was made to determine the value of b from the measured sediment transport rates during the various experiments. Fig. 5.2-2a presents the sediment transport rates per unit width versus the average velocity as measured during the different experiments.

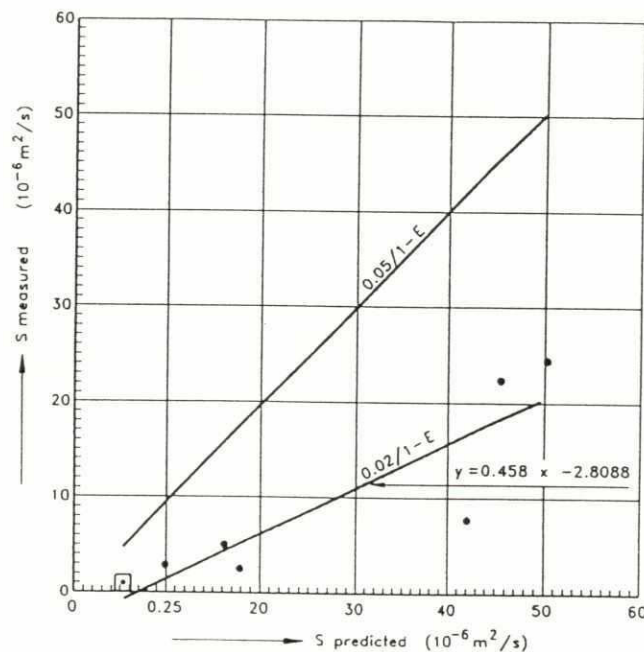
From the figure it was not possible to derive the power accurately due to the substantial scatter (see also Section 2.2.3). Even if one other point was added as obtained from

Struiksmas & Crosato (1989), who used the same sand, the picture was still obscure. The best fitting line corresponded to about $b = 3.8$. In another attempt the measured sediment transport was plotted versus predicted values using the Engelund-Hansen (1979) sediment transport predictor, which reads as:

$$s = \frac{0.05}{1-\epsilon} \sqrt{g\Delta D} \left(\frac{h_i}{\Delta D} \right)^{5/2} \frac{C^2}{g} \quad (5.2-1)$$



a. measured sediment transport rates per unit width versus the average velocity



b. measured sediment transport versus predicted values using E & H predictor

Fig. 5.2-2: An attempt to find the power b

The results are given in Fig. 5.2-2b. It can be concluded that Engelund-Hansen predictor yields a fair prediction of the sediment transport of the fine bed material in the experimental facility used, if the constant 0.05 is changed to 0.02. Thus it was assumed for the time being that a power of 5 is a good approximation for the power of the sediment transport formula. However, a slightly lower value (down to say 4) can also be applicable.

The value of b being fixed (temporarily) and only the values λ_w , λ_s and their ratio are of importance. These two parameters were computed for the various experiments and they were used to plot the graph for each experiment with a curved channel as defined in Section 2.2.6:

$$H = H_0 e^{-\frac{x'}{L_p}} \sin \left(2\frac{\pi}{L_p} x' + \phi \right) \quad (5.2-2)$$

It was assumed that $x' = 0$ corresponds to the beginning of the bend and that $\phi = -\pi/2$.

The results of computed and measured H/H_0 are plotted in Figs. 5.2-3 through 5.2-5, where the value of the power b for computed ones varied between 4 and 5. The value of the parameter E was kept 0.5 (also used by Struiksma and Crosato, 1989). Since the channel was too short to reach the axisymmetric condition, so for drawing Fig. 5.2-3b through 5.2-5-b, H_0 was estimated from equation (2.2-59) without effect of bank erosion.

$$\pi \frac{H_0}{B_0} = Af(\theta) \frac{h_0}{R_c} \quad (5.2-3)$$

and H_x (effect of overshoot phenomena) was computed from:

$$H_{(x)} = h_{(x)} - h_o - H_0 \quad (5.2-4)$$

which in this relation $h_{(x)}$ is measured water depth at outer bend.

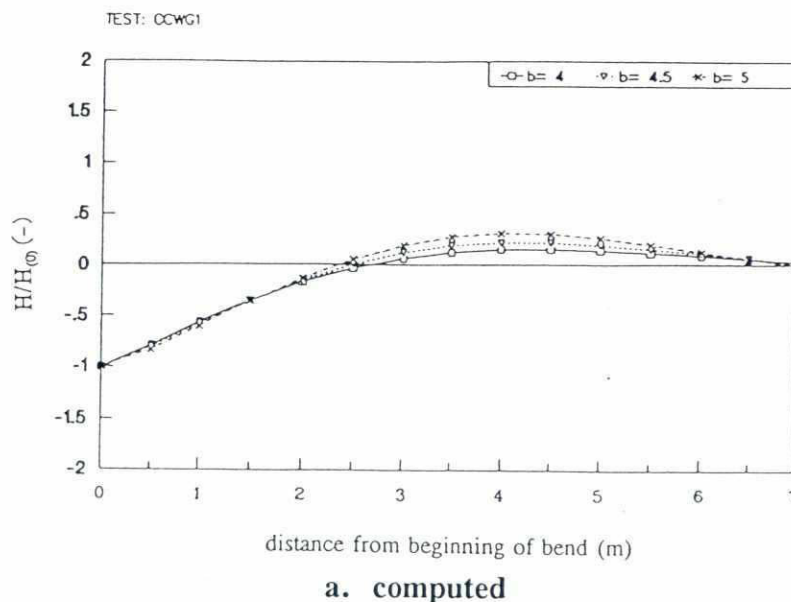
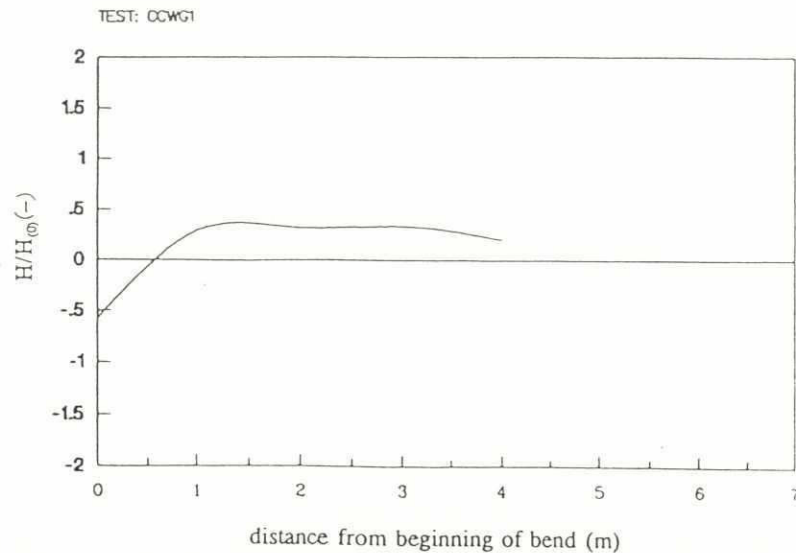


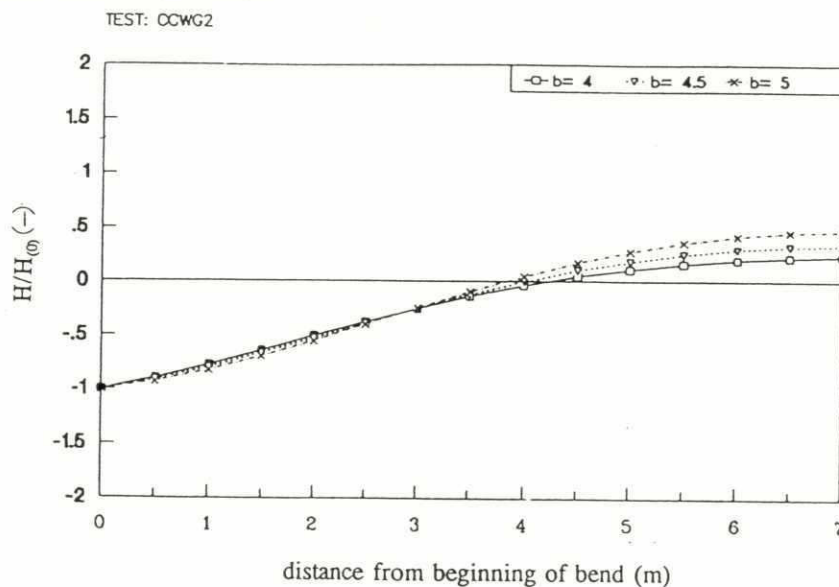
Fig. 5.2-3: Dimensionless water depth along outer bend (CCWG1)



b. measured

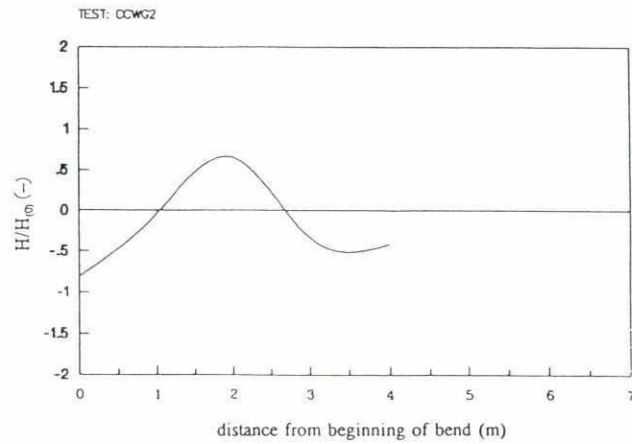
Fig. 5.2-3 (continued): Dimensionless water depth along outer bend (CCWG1)

One of the things that is observed is the location of the maximum overshoot which is much more downstream than observed in the experiments. This location can be influenced by the selection of the power b and by the parameter E . From Fig. 5.2-3 through 5.2-5, it is clear that the power b influences mainly the dampening and not so much the location of the deepest outer bend scour. The coefficients are more effective in this respect. This is shown in Fig. 5.2-6a. To shift the location of the deepest point, very high values of E have to be selected and this is not in line with other observations in flumes (see Struiksma et al., 1985 and Struiksma and Crosato, 1989).



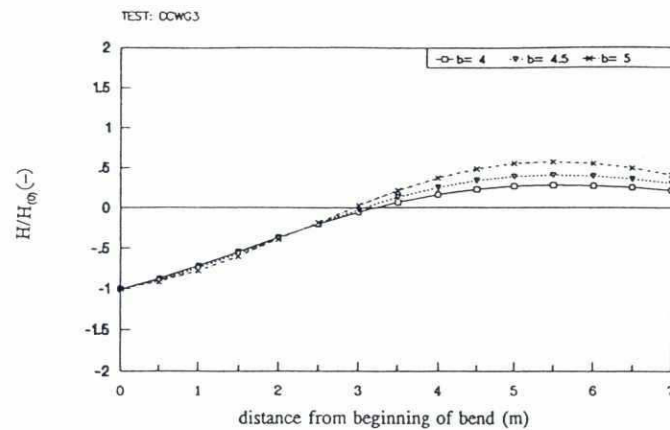
a. computed

Fig. 5.2-4: Dimensionless water depth along outer bend (CCWG2)

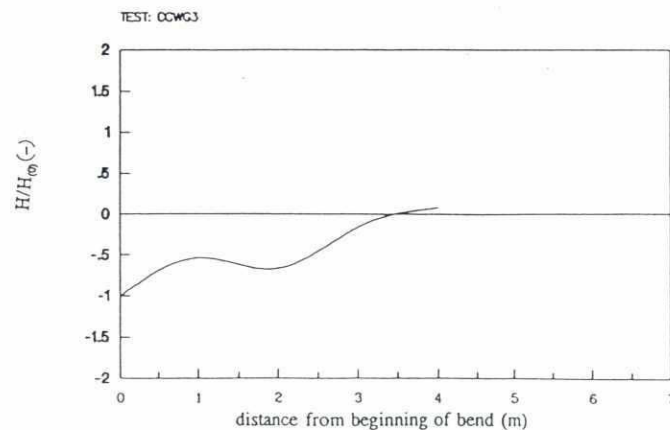


b. measured

Fig. 5.2-4 (continued): Dimensionless water depth along outer bend (CCWG2)



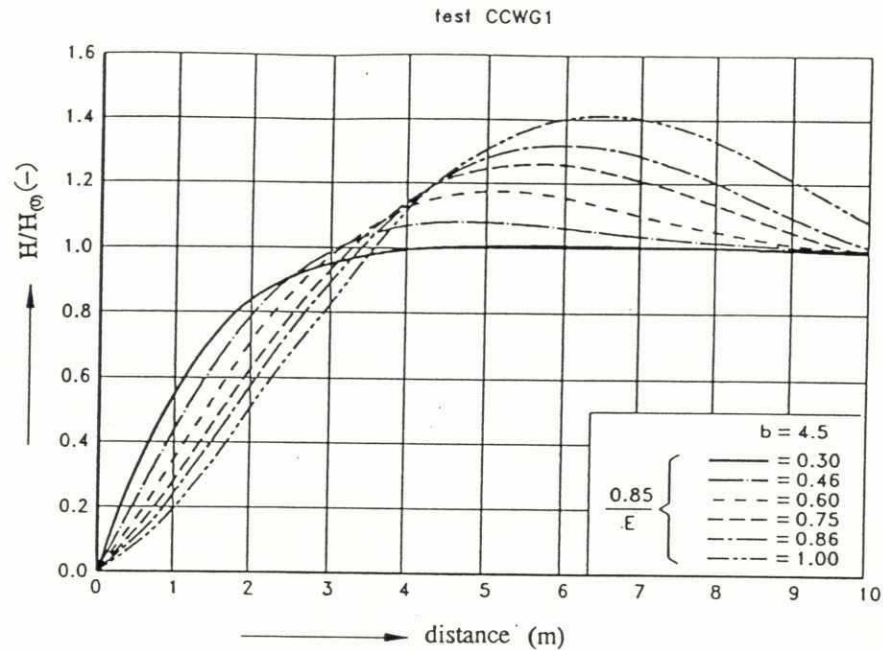
a. computed



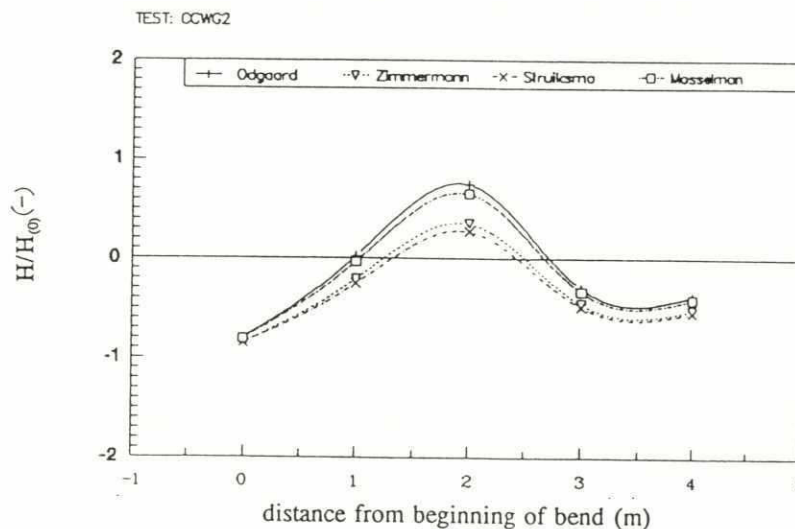
b. measured

Fig. 5.2-5: Dimensionless water depth along outer bend (CCWG3)

Also from Figs. 5.2-3 through 5.2-5 it is clear that the wave length of flow in measured graph is much more shorter than the computed ones (specially in test CCWG2). In another attempt, the value of H_o in test CCWG2 was estimated by using the methods of Odgaard (1984), Zimmermann and Kennedy (1979) and Struiksma (1988) for axisymmetric condition and the results were compared with the results of measured ratio of $H_{(x)}/H_o$ based on the equations (5.1-1), (5.1-2) and Fig. 5.2-6b. This figure shows that there is not any difference in location of overshoot, but the value of it is changed.



a. $\frac{0.85}{E} = \text{varying between } 0.30 - 1.00$



b. measured graph based on different methods

Fig. 5.2-6: Variation of computed dimensionless overshoot

Now the results of the tests in axisymmetric conditions are being discussed. Theoretically, the characteristics of bend flow and bed topography in the axisymmetric conditions of bends were explained in Section 2.2.5. Although in the conducted tests, due to the length of the channel, these conditions were not present, but here an attempt has been made to check the results of the test with the various predictors based on the fully developed bend flow. If one considers Table 5.2-2, Fig. 5.2-7 and Fig. 5.2-8 related to the various cross-sections of the first and second series, it is possible to determine the effect of spiral motion of bend flow on the bed topography and scour in the outer bend.

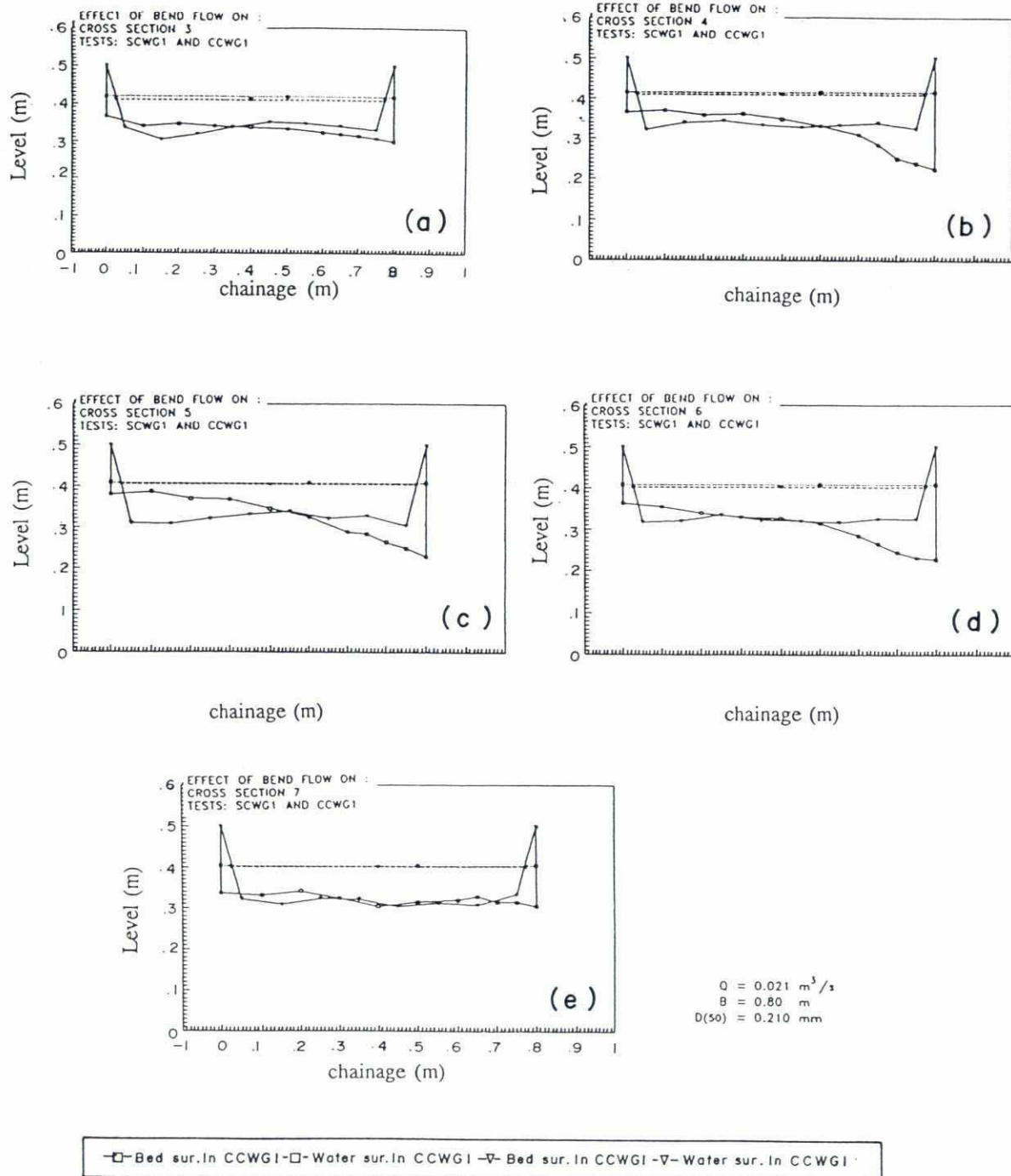


Fig. 5.2-7: Effect of bend flow on the cross-sections, first series

From Table 5.2-1 it is obvious that the relative bend scour increases gradually from the beginning of the bend towards the middle and then decreases from the middle towards the end of the bend in both series. They show an overshooting along the bend of which the apex is around the middle of the bend, see also Fig. 5.2-9.

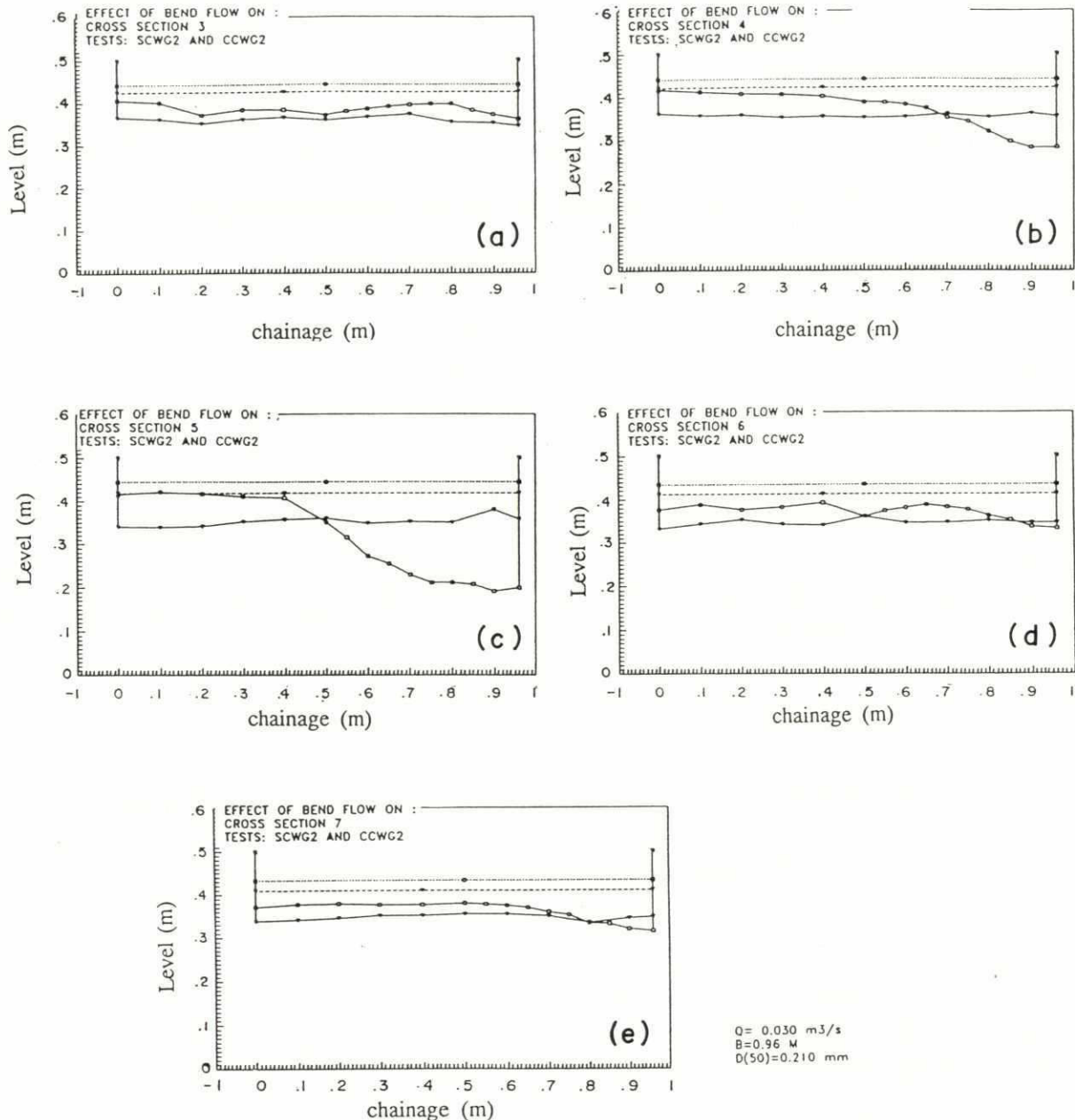
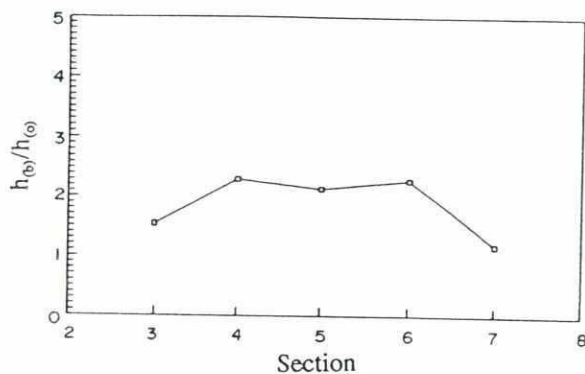


Fig. 5.2-8: Effect of bend flow on the cross-sections, second series

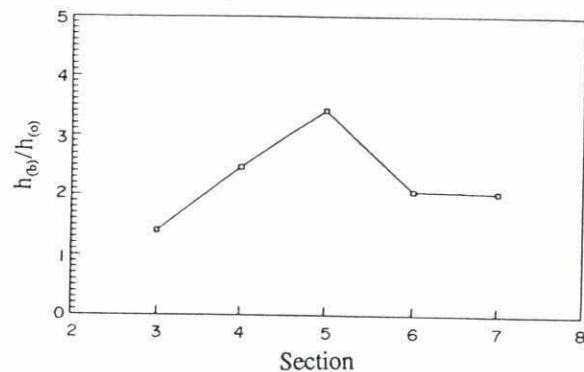
Now the results of tests CCWG1 & CCWG2 are compared with the various methods for predicting the maximum bend scour which were described in Section 2.2.4 (Odgaard, Zimmermann and Struiksmma methods).

Test: Section	First Series:			Second Series:		
	SCWG1 $h(o)$	CCWG1 $h(b)$	$\frac{h(b)}{h(o)}$	SCWG2 $h(o)$	CCWG2 $h(b)$	$\frac{h(b)}{h(o)}$
3	.077	.118	1.532	.066	.093	1.409
4	.077	.177	2.299	.066	.163	2.470
5	.084	.179	2.131	.066	.225	3.409
6	.079	.18	2.278	.066	.136	2.061
7	.84	.099	1.179	.063	.129	2.048

Table 5.2-1: Effect of bend flow in first and second series



a. first series



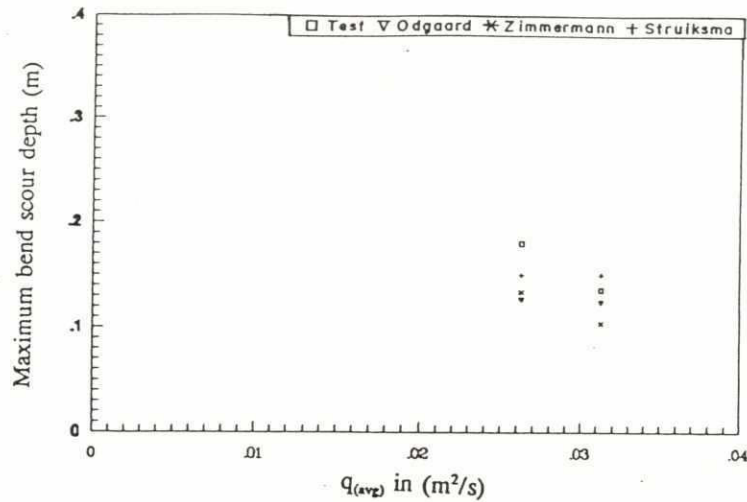
b. second series

Fig. 5.2-9: Longitudinal bed profile near the outer bank

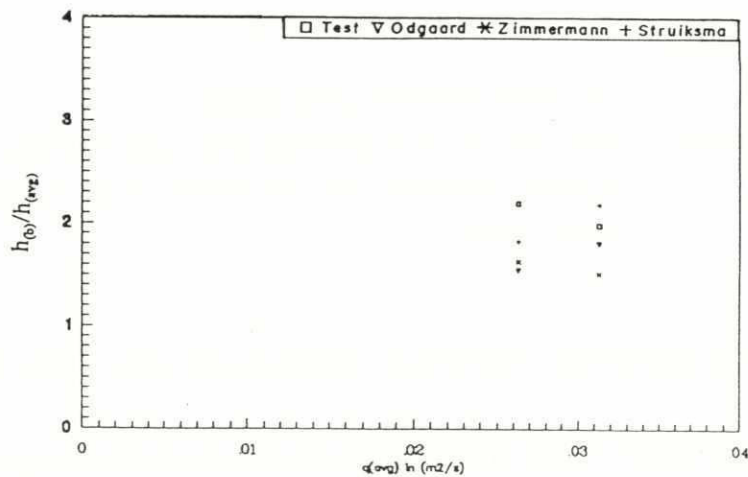
Since these methods are based on the fully developed flow in bend, the comparison is done considering the parameters of cross-section 6 (near the end of bend) and also based on the average parameters of the tests, see Fig. 5.2-10 and 5.2-11. In Fig. 5.2-10a the maximum bend scour depth of both the tests (at cross-section 6) are compared with those calculated by the various methods and in Fig. 5.2-10b, this comparison is done on the relative bend scour (h_b/h_{avg}). In Fig. 5.2-11a and 5.2-11b the same comparisons are performed but based on the average parameter of the tests.

From the figures the following results can be obtained:

- the results of the predictors are lower than the test results;
- the results of Odgaard and Zimmerman methods are in close agreement;
- the result of the Struiksmas graph is closer to the test result, and
- by increasing the discharge intensity, the dispersions of the results becomes greater.

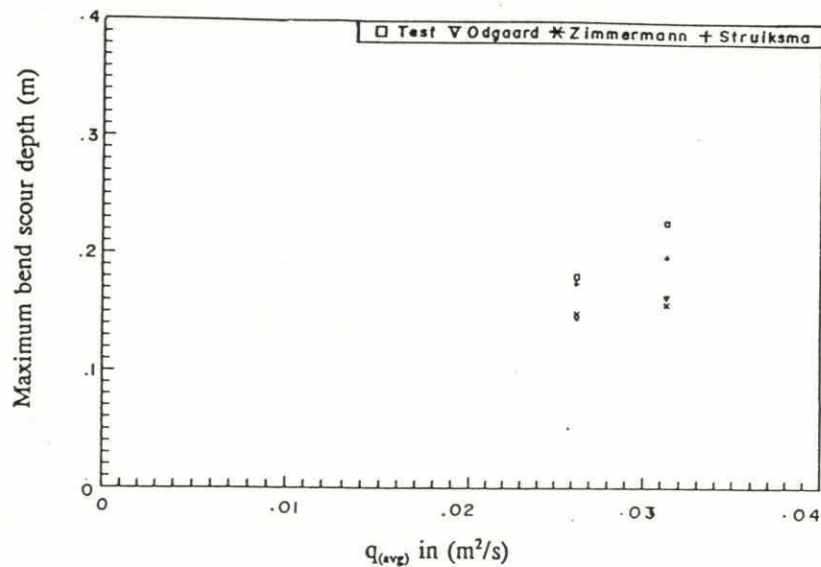


a. maximum bend scour depth



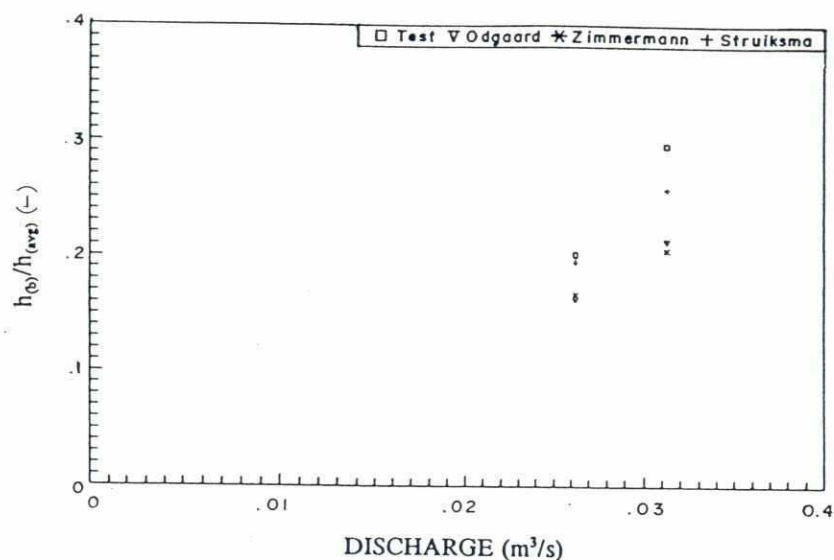
b. relative bend scour depth

Fig. 5.2-10: The comparison between the results of the tests and the various methods on the bend scour based on the parameters of section 6



a. maximum bend scour depth

Fig. 5.2-11: The comparison between the results of the tests and the various methods on the bend scour based on average parameters



b. relative bend scour depth

Fig. 5.2-11 (continued): The comparison between the results of the tests and the various methods on the bend scour based on average parameters

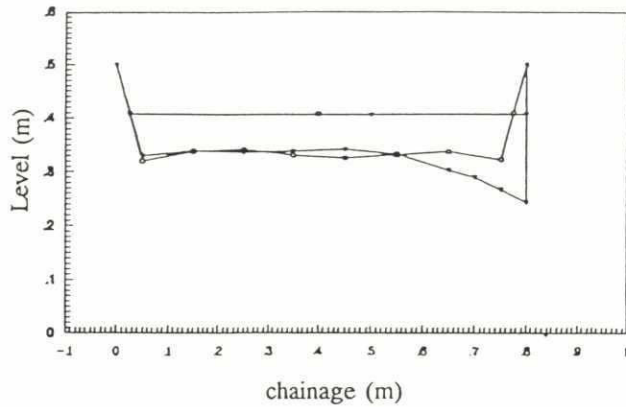
5.3 LOCAL SCOUR AROUND GROYNES - STRAIGHT CHANNEL TESTS

As was explained in Section 2.3, one of the main purposes of constructing groynes along the banks of channels is the protection of the banks, but it is obvious that a scour hole will be created around the head of each groyne. The depth of this scour hole should be considered in relation to the stability of the groyne.

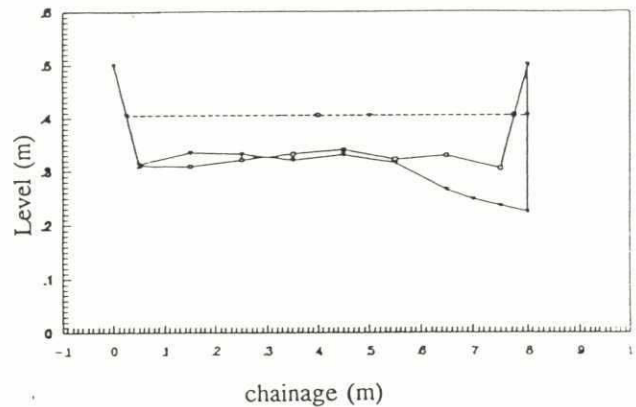
The tests carried out in this study confirm that the groynes have a considerable effect on erosion of the river bed around the head of groynes. Table 5.3-1 and Figs. 5.3-1 and 5.3-2 show this effect.

Test: section	First Series:			Second Series:		
	SCWG1 $h(o)$	SCIG1.2 $h(l)$	$\frac{h(l)}{h(o)}$	SCWG2 $h(o)$	SCIG2 $h(l)$	$\frac{h(l)}{h(o)}$
3	.077	.095	1.234	.066	.125	1.894
4	.077	.164	2.130	.066	.121	1.833
5	.084	.180	2.143	.066	.119	1.803
6	.079	.181	2.291	.066	.131	1.985
7	.084	.171	2.036	.063	.156	2.476

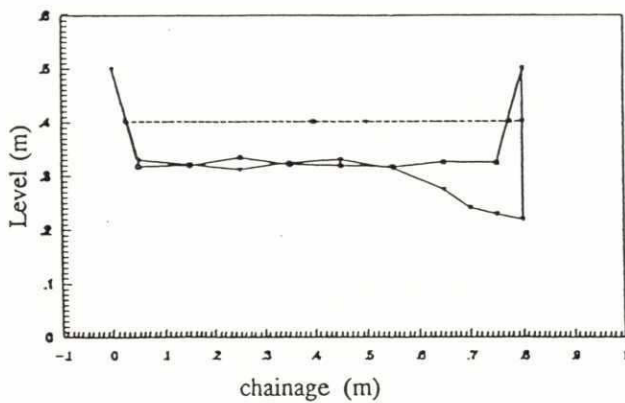
Table 5.3-1: Relative local scour depth in front of groynes in the first and second series (values of water depths in meter)



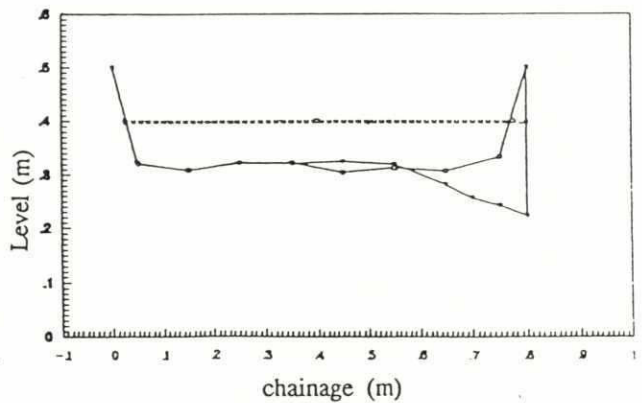
a. Section 4



b. Section 5



c. Section 6

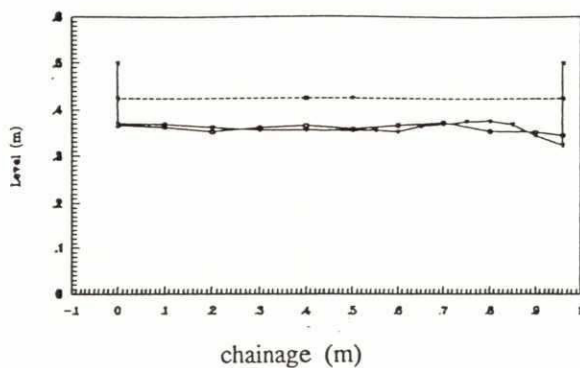


d. Section 7

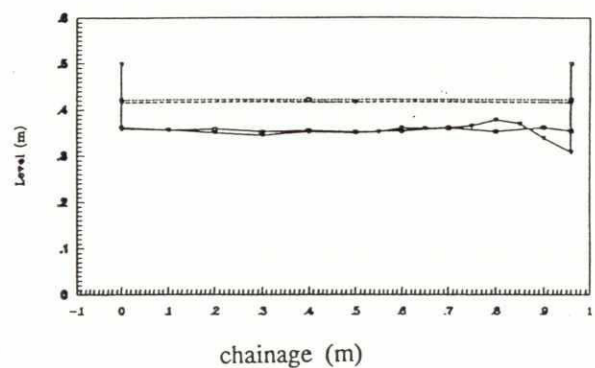
—□— Bed sur.(SCIGL2) —□— Water sur.(SCIGL2) —▽— Bed sur.(SCIGL2) —▽— Water sur.(SCIGL2)

$Q = 0.021 \text{ m}^3/\text{s}$
 $B = 0.80 \text{ m}$
 $D(50) = 0.210 \text{ mm}$

Fig. 5.3-1: Effect of groynes in straight channel, cross-sections of first series

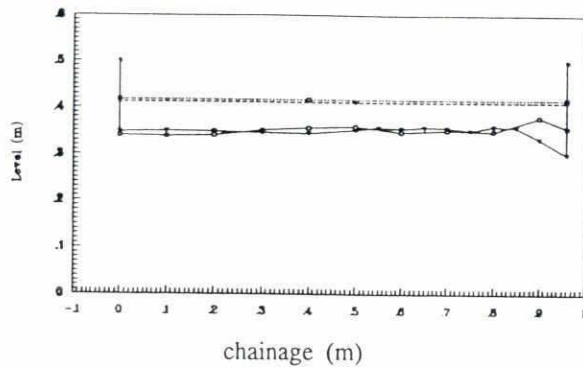


a. Section 3

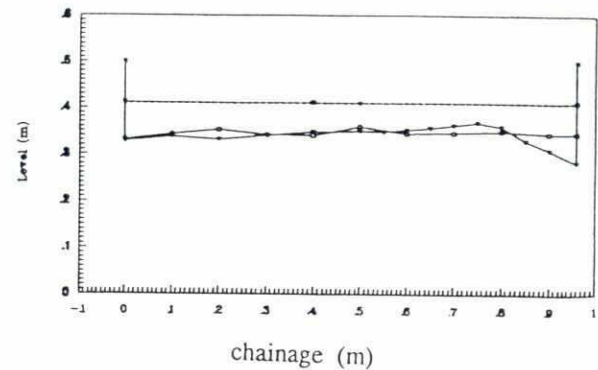


b. Section 4

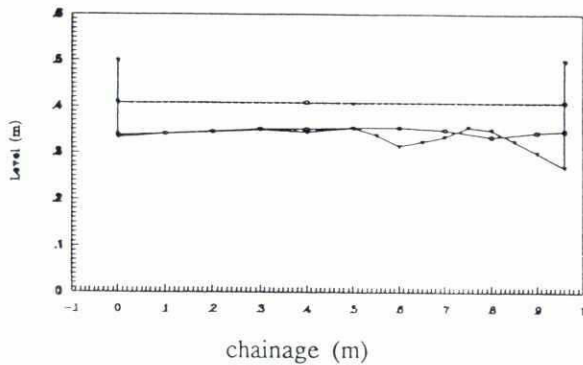
Fig. 5.3-2: Effect of groynes in straight channel, cross-sections of second series



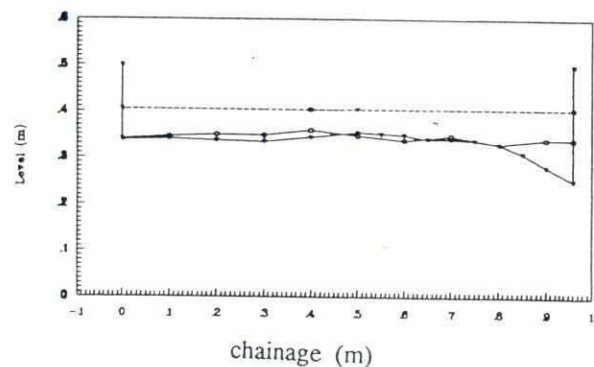
c. Section 5



d. Section 6



e. Section 7



f. Section 8

$$Q = 0.030 \text{ m}^3/\text{s}$$

$$B = 0.95 \text{ m}$$

$$D(50) = 0.210 \text{ mm}$$

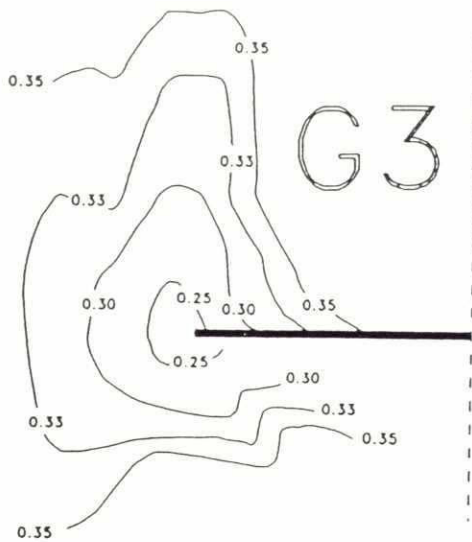
□ Bed sur. (without bank ero.) □ Water sur. (without bank ero.) ▽ Bed sur. (with bank ero.) ▽ Water sur. (with bank ero.)

Fig. 5.3-2: Effect of groynes in straight channel, cross-sections of second series

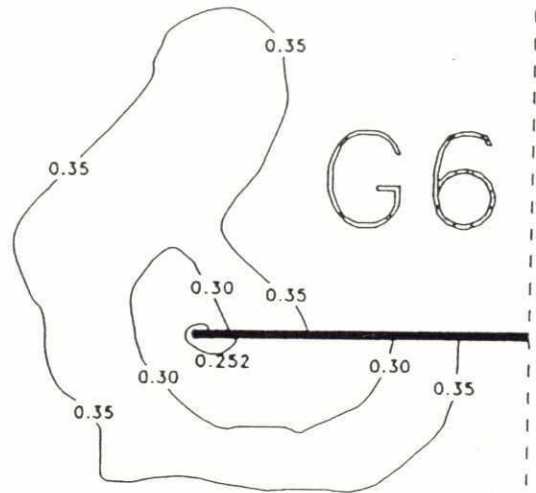
From tests SCIG1.2 and SCIG2 it is clear that the maximum scour depth of the scour hole occurs in front of the groynes. Fig. 5.3-3 shows two scour holes around the groyne 3 of test SCIG1.2 and the groyne 6 of test SCIG2.

The shape of scour holes seem horse shoe like tending towards the downstream side of the groyne, and showing a decreasing scour depth from the nose of the groyne to its stem. Also, sand deposition occurs in the pocket between the groynes, due to a large but weak eddy which exists in the pocket. On the other side, because of two small but strong eddies before and after the groynes, bank erosion occurs near the groynes. Photo 7 shows the effects of these eddies.

It should be mentioned that the dimensions of the scour holes around the groynes are not scaled correctly compared with reality, due to large scale effects induced by the distortion.



a. Test SCIG1.2



b. Test SCIG2

Fig. 5.3-3: Scour holes around the groyne

To consider the increase of the maximum scour depth at the head of the groynes in the downstream direction of the channel, what is shown in Fig. 5.3-4, can be interesting. Therefore, the location of the groynes along the channel has also some impact on the local scour depth of the groynes, and should be accounted for in the design of spurs.

Next a comparison is made between the results of the conducted tests and several predictors of the maximum local scour depth in front of groynes such as Ahmad, Lacey, Garde and Gill predictors as explained in Section 2.3.3. To do this comparison, the average parameters of tests SCIG1.2 and SCIG2 are used. The results of the tests are compared in two cases, dimensional and dimensionless maximum local water depth in front of groynes in the channel, Fig. 5.3-5 shows this comparison.

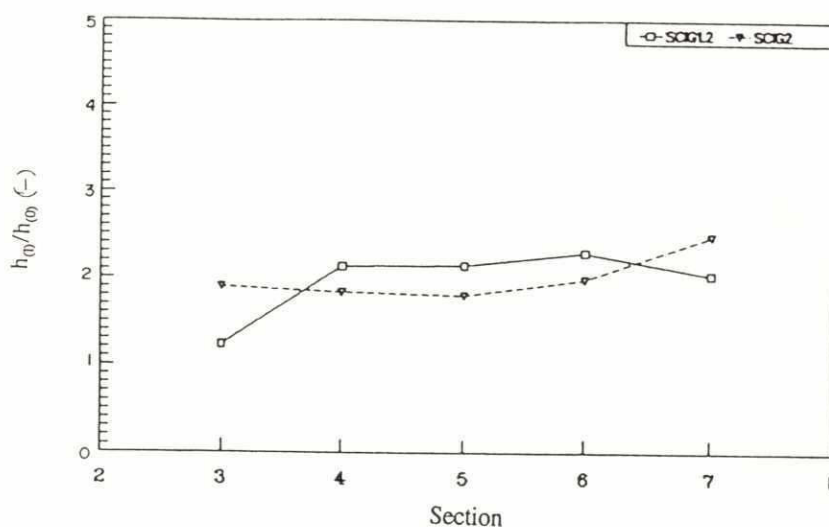
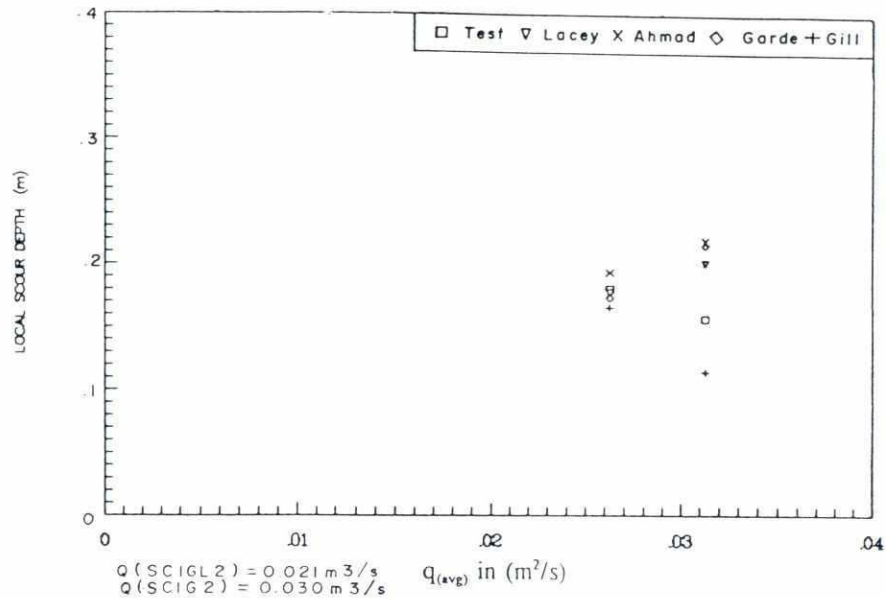


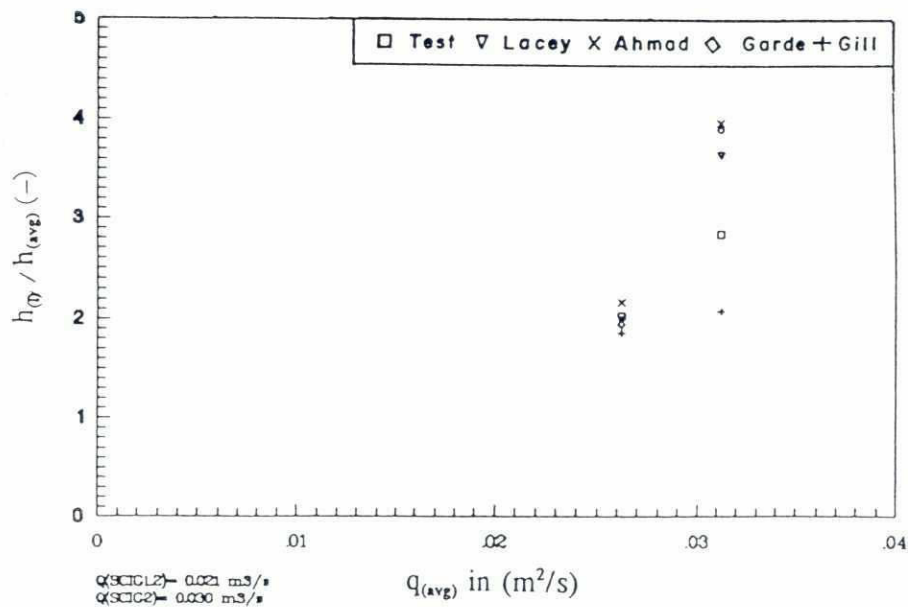
Fig. 5.3-4: Maximum scour depth at the nose of spurs along the channel of tests SCIG1.2 & SCIG2

From this figure, the following facts can be inferred:

1. The predictors can give a good estimation, particularly in the range of low discharge intensity.
2. When increasing the discharge, Ahmad, Lacey and Garde et al. give an over-estimation of maximum scour depth and Gill's prediction gives an underestimation.
3. Ahmad, Lacey and Garde et al. predictors are in good agreement with each other.



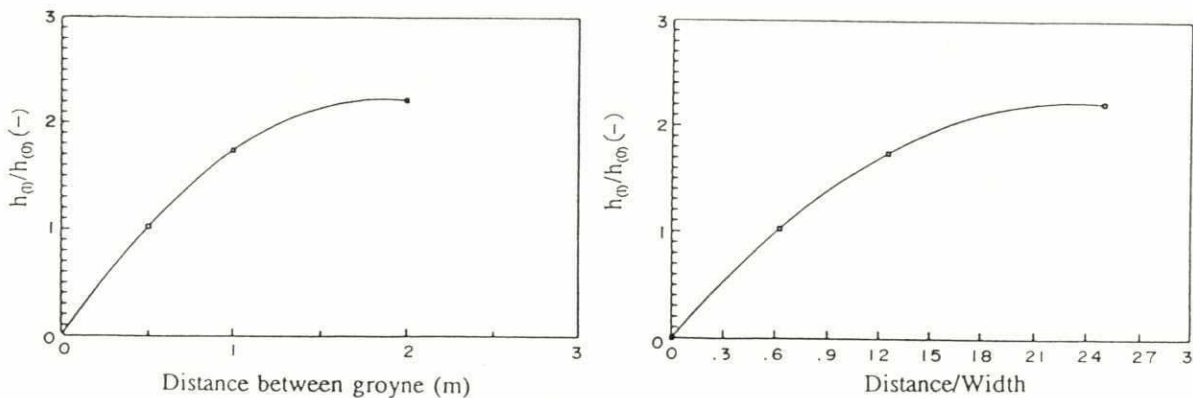
a. dimensional water depth



b. dimensionless water depth

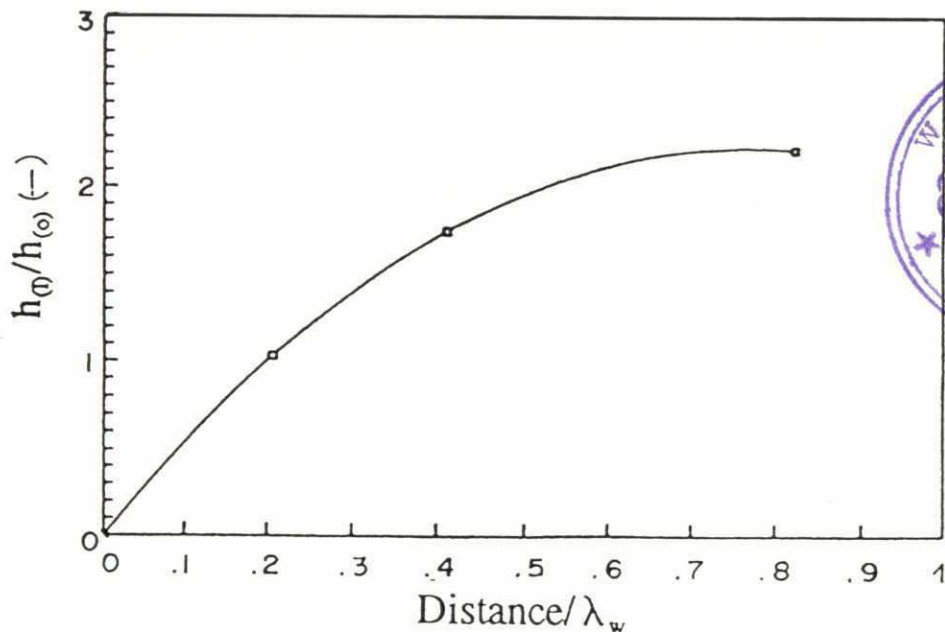
Fig. 5.3-5: Comparison of the tests results with the predictors

During the first series of tests, an additional study was performed to find the effect of spacing of the groynes on the maximum local scour depth. For this purpose test SCIG1 was conducted in three parts with distances 0.5, 1 and 2 m between the groynes, respectively. The dimensionless relative maximum water depth in front of the groynes in three cases are plotted against (a) distance of groynes, (b) the ratio of distance of groynes to width of channel and (c) the ratio of distance of groynes to the relaxation length (λ_w) of the flow, as shown in Fig. 5.3-6. The result of this study is satisfactory and the graphs seem reliable and logical.



a. h_l/h_0 versus distance of groynes

b. h_l/h_0 versus distance/width



c. h_l/h_0 versus distance/ λ_w

Fig. 5.3-6: Effect of distance of groynes on the maximum local scour depth

5.4 COMBINED BEND AND LOCAL SCOUR

In Sections 5.2 and 5.3 the bend scour in curved channels and the local scour in front of groynes in straight channels were discussed separately. Each one has a significant influence on the bed topography of the curved channel when the groynes are used along the outer bank. There is a complex situation in the bend which is affected by the various phenomena. However, it is obvious that constructed groynes in bends create deeper scour in outer bends. This effect can be identified in Table 5.4-1 and Fig. 5.4-1 and 5.4-2. The figures present the average water depth in test SCWG (h_o) and the maximum water depths along the channel in tests SCIG (h_i), CCWG (h_b) and CCIG(h_{lb}) for the first and second series, respectively.

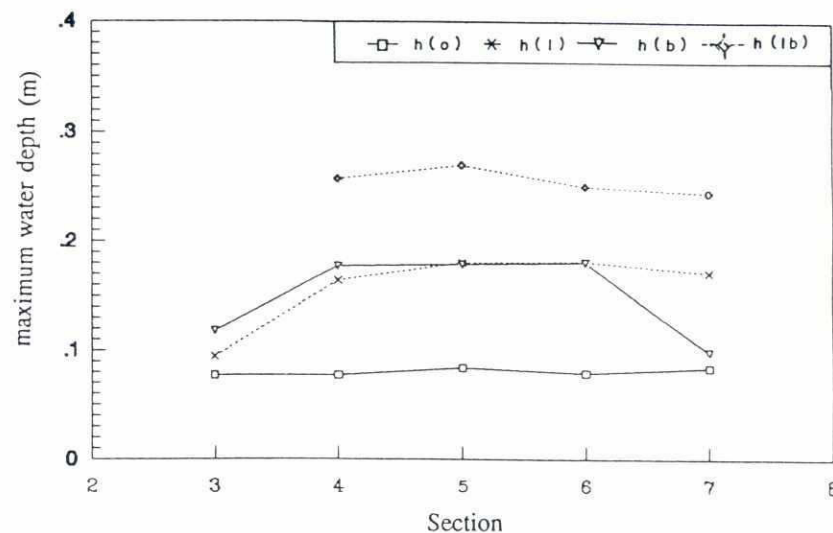


Fig. 5.4-1: Various water depths in the first series

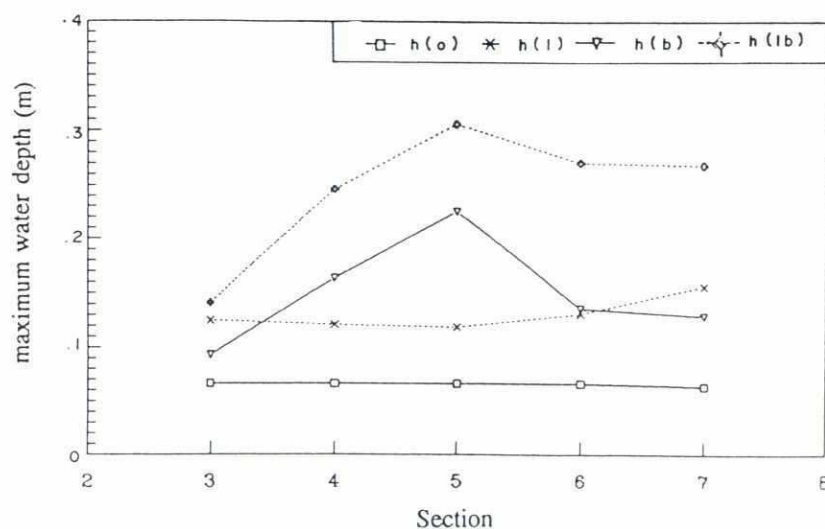


Fig. 5.4-2: Various water depths in the second series

In spite of the complexity of the scouring phenomena in the curved channel with the groynes, and also the lack of data in this case, the Consultant tried to find a relation between h_o , h_l , h_b and h_{lb} in a simple way, because Figs. 5.4-1 and 5.4-2 denote a harmonious behaviour between these two. To find this relation, the various values are calculated based on these four different water depths and these are inserted in Table 5.4-1 to be available for more study.

a. first series:									
$\Delta h(l)$	$\Delta h(b)$	$\Delta h(l+b)$	$\Delta h(lb)$	$\frac{h(l)}{h(o)}$	$\frac{h(b)}{h(o)}$	$\frac{h(lb)}{h(o)}$	$\frac{\Delta h(l)}{h(o)}$	$\frac{\Delta h(b)}{h(o)}$	$\frac{\Delta h(lb)}{h(o)}$
.018	.041	.059	-	1.234	1.532	-	.234	.532	-
.087	.100	.187	.180	2.130	2.299	3.338	1.130	1.299	2.338
.096	.095	.191	.186	2.143	2.131	3.214	1.143	1.131	2.214
.102	.101	.203	.171	2.291	2.278	3.165	1.291	1.278	2.165
.087	.015	.102	.160	2.036	1.179	2.905	1.036	.179	1.905
b. Second series									
$\Delta h(l)$	$\Delta h(b)$	$\Delta h(l+b)$	$\Delta h(lb)$	$\frac{h(l)}{h(o)}$	$\frac{h(b)}{h(o)}$	$\frac{h(lb)}{h(o)}$	$\frac{\Delta h(l)}{h(o)}$	$\frac{\Delta h(b)}{h(o)}$	$\frac{\Delta h(lb)}{h(o)}$
.059	.027	.086	.075	1.894	1.409	2.136	.894	.409	1.136
.055	.097	.152	.180	1.833	2.470	3.727	.833	1.470	2.727
.053	.159	.212	.239	1.803	3.409	4.621	.803	2.409	3.621
.065	.070	.135	.204	1.985	2.061	4.091	.985	1.061	3.091
.093	.066	.159	.205	2.476	2.048	4.254	1.476	1.048	3.254

Table 5.4-1: Various values based on h_o , h_l , h_b and h_{lb}

Studying Table 5.4-1, it is concluded that the best relation between the various water depths is the equation (5.4-1) which reads as:

$$h_{(lb)} \cong h_{(o)} + \Delta h_{(l)} + \Delta h_{(b)} \quad (5.4-1)$$

in which

$$\Delta h_{(l)} = h_{(l)} - h_{(o)} \quad (5.4-2)$$

$$\Delta h_{(b)} = h_{(b)} - h_{(o)} \quad (5.4-3)$$

Based on equation (5.4-1), Fig. 5.4-3 and 5.4-4 have been prepared, and they indicated a near agreement between $h_{(lb)}$ and h_{total} which is $h_o + \Delta h_l + \Delta h_b$.

Finally, the results of tests CCIG1 and CCIG2 are compared with the only available predictors from Ahmad and Lacey which were explained in Section 2.4.2. Since Ahmad and Lacey introduced a range for coefficient (K) of their formulas, based on the position of bend and the groynes along the bend, this comparison is performed between the minimum and maximum water depth at the head of groynes in the tests, and these are calculated by the predictors while considering the following ranges:

Ahmad $2.2 < K < 2.6$ and Lacey $1.5 < K < 1.75$; as represented in Fig. 5.4-5.

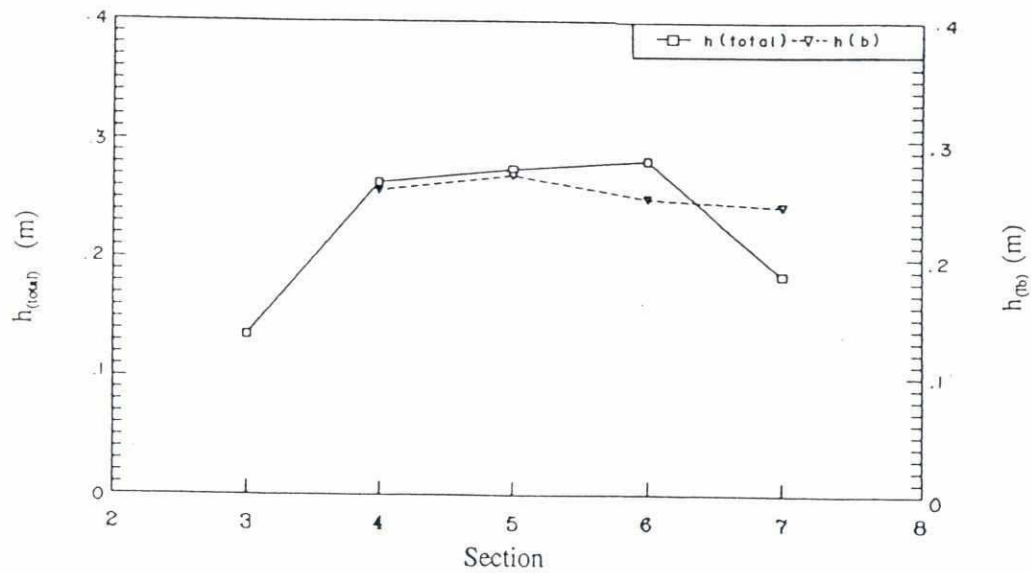


Fig. 5.4-3: Comparing h_{total} with h_{lb} in first series

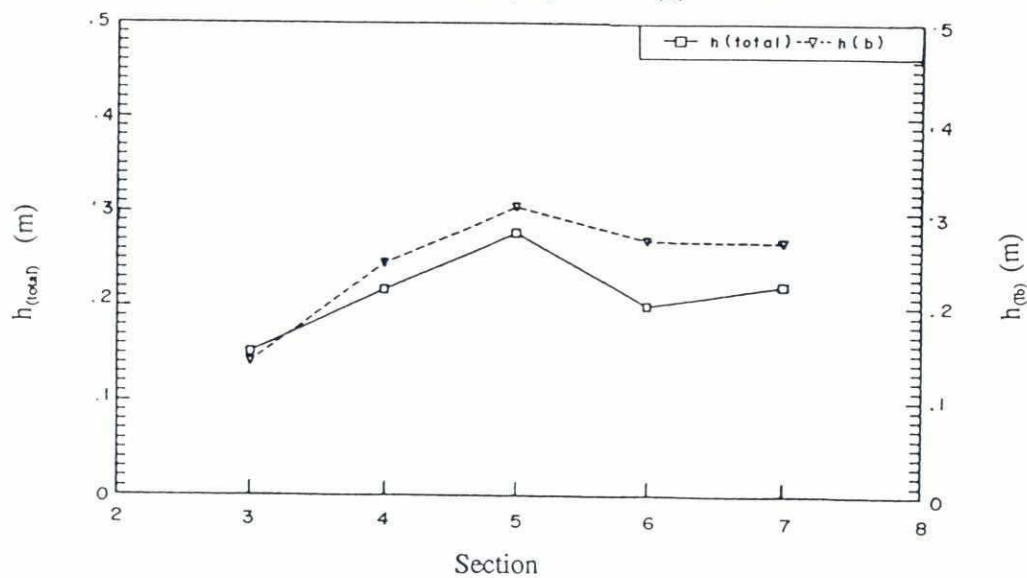
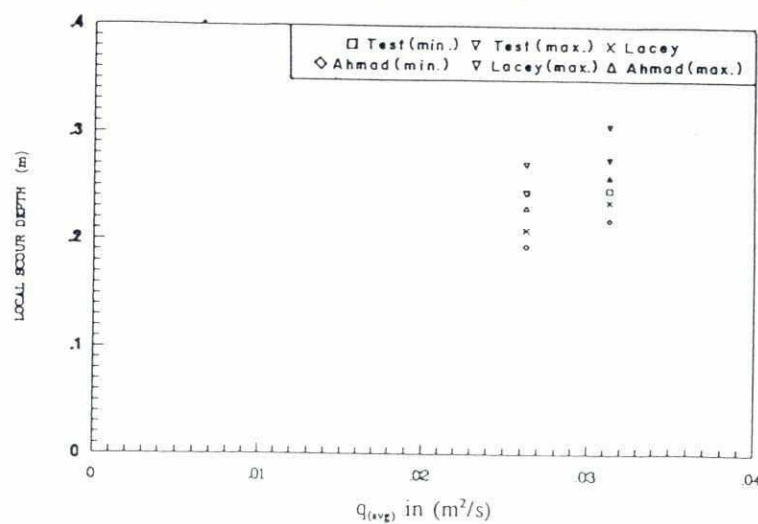
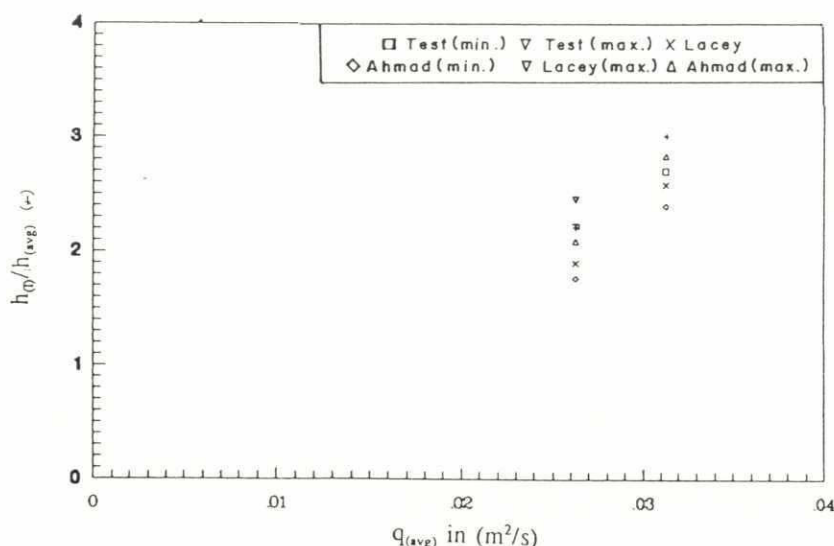


Fig. 5.4-4: Comparing h_{total} with h_{lb} in second series



a. dimensional maximum local scour depth

Fig. 5.4-5 : Comparing the result of tests with the predictors on maximum local scour depth in curved channel



b. dimensionless maximum local scour depth

Fig. 5.4-5 (continued): Comparing the result of tests with the predictors on maximum local scour depth in curved channel

From this comparison, it can be concluded that there is a satisfactory agreement between the results of the tests with Ahmad and Lacey predictors.

5.5 INFLUENCE OF BANK EROSION PRODUCTS ON BED SLOPE

The input of bank erosion products reduces transverse bed slopes in the bends, which is the result obtained by Mosselman (1989), but it was concerned mostly on the characteristics of cohesive material (Section 2.2.7).

In this study, the cohesionless material (sand with $D_{50} = 0.210$ mm) was used and three experiments were conducted, as explained in Section 4.2, but only the results of the first and second tests are acceptable and useful.

In both experiments CCWIG1 and CCWIG2, the distance between the groynes and fixed outer bank (in first part of tests) was 0.30 m. This amount of sand in the middle of bend was eroded during 60 minutes in test CCWIG1 and 15 minutes in the second test CCWIG2. So the average rate of erosion can be calculated as follows.

Test CCWIG1:

$$\frac{\partial n_1}{\partial t_1} = \frac{0.30}{60 * 60} = 8.3 * 10^{-5} \quad \text{m/s} \quad (5.5-1)$$

Test CCWIG2:

$$\frac{\partial n_2}{\partial t_2} = \frac{0.30}{15 * 60} = 3.3 * 10^{-4} \quad \text{m/s} \quad (5.5-2)$$

The increase in the rate of erosion of the second test is reasonable due to higher discharge and steeper slope.

Fig. 5.5-1 clearly show the influence of bank erosion products on the bed slope of the first part of the test and just before the effect of groynes which created a scour hole around the head of each groyne. The figures confirm the result of Mosselman and on an average the input of bank erosion decreased the maximum bend scour depth by about 50 %, or in other words, the bed level rose (see Table 5.5-1).

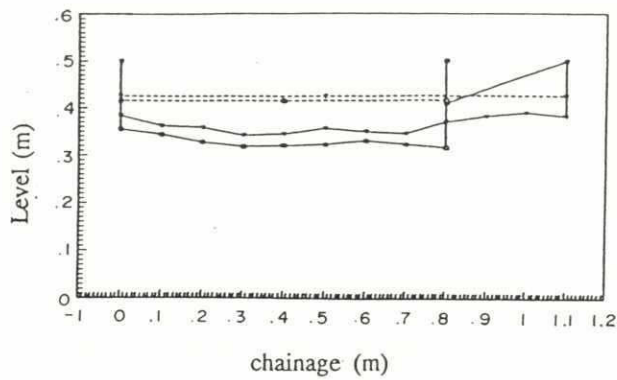
In Table 5.5-1, $h_{(b1)}$ is the maximum water depth near the outer bank before removing the fixed plates, $h_{(b2)}$ is the water depth in the same location of cross-section after effect of bank erosion and $\Delta h_{(b)}$ is the difference of $h_{(b1)}$ and $h_{(b2)}$.

Fig. 5.5-2 indicates the changes of the bed level in the various cross-sections after the effect of bank erosion and the groynes installed at 0.30 m away from the outer bank in the first part, and in the equilibrium conditions of channel test CCWIG2.

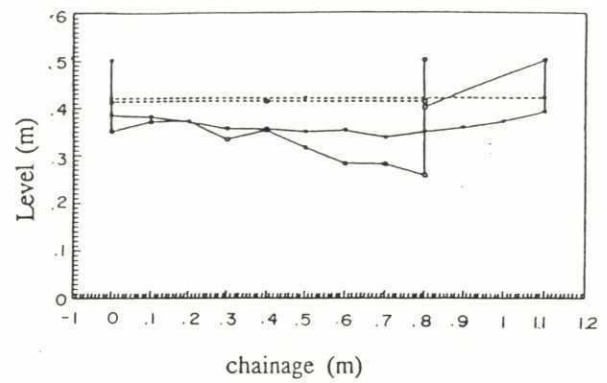
Section	$h_{(b1)}$ (m)	$h_{(b2)}$ (m)	$\Delta h_{(b)}$ (m)	$\frac{\Delta h_b}{h_{(b1)}}$ (%)
3	0.101	0.056	0.045	44.55
4	0.156	0.071	0.085	54.49
5	0.185	0.081	0.104	56.21
6	0.128	0.077	0.051	39.84
7	0.119	0.052	0.067	56.30
Average				50.278

Table 5.5-1: Percentage of the influence of bank erosion on the maximum bend scour depth (Test CCWIG1)

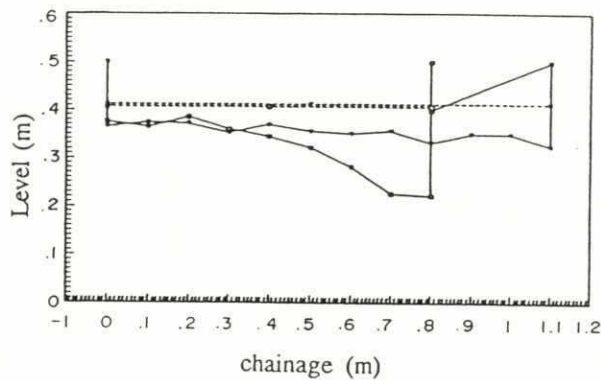
It is clear that the groynes reduced the bed level of the channel in front of the groynes, but the sand materials were deposited on the bed of the channel between the groynes and in total along the channel, meaning both a rise of the bed level and increase in bed slope. In this test the effect of bank erosion and the groynes on the bed level for the various cross-sections was different in time. So Table 5.5-2 includes the maximum effect of bank erosion on the bed level over various periods during the tests related to every cross-section.



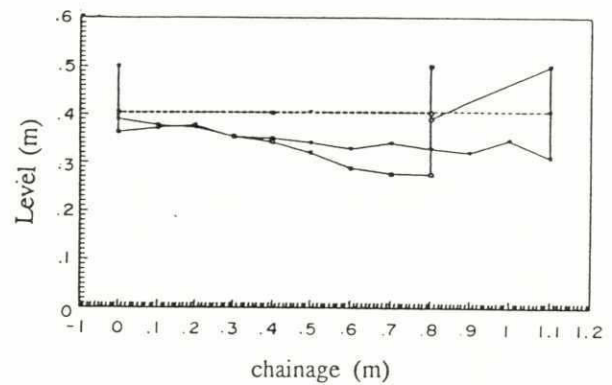
a. Section 3



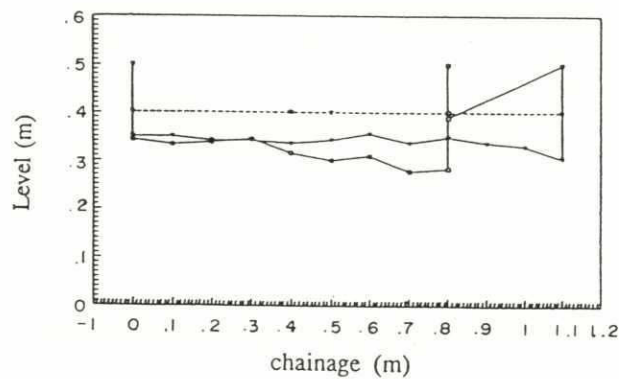
b. Section 4



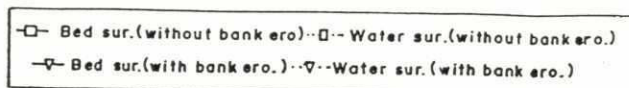
c. Section 5



d. Section 6

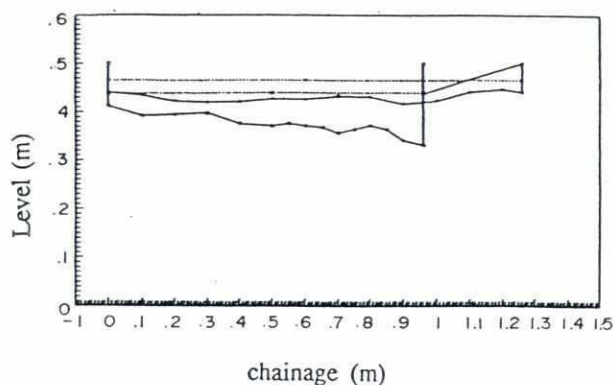


e. Section 7

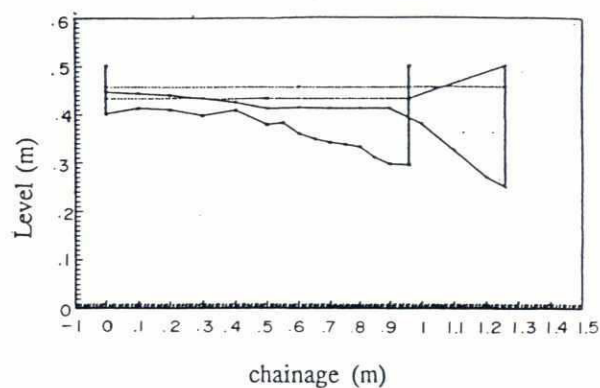


$Q = 0.0021 \text{ m}^3/\text{s}$
 $B = 0.8 \text{ m}$

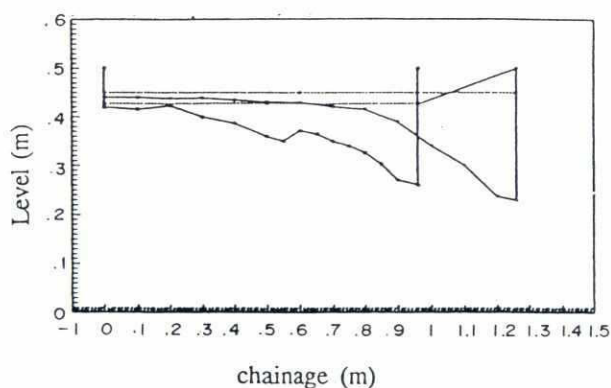
Fig. 5.5-1: Effect of bank erosion on the bed slope of bend (Test CCWIG1)



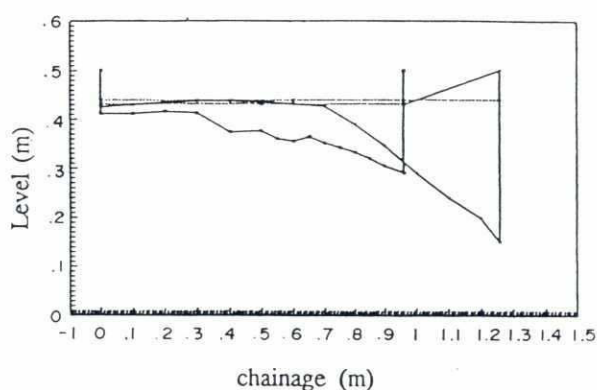
a. Section 3



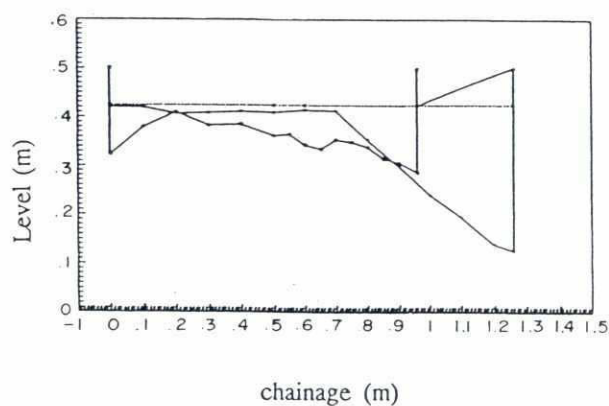
b. Section 4



c. Section 5



d. Section 6



e. Section 7

$Q = 0.030 \text{ m}^3/\text{s}$
 $S = 0.003$
 $B(\text{initial}) = 0.96 \text{ m}$

—□— Bed sur.(without bank ero.) —○— Water sur.(without bank ero.) —△— Bed sur.(with bank ero.) —◇— Water sur.(with bank ero.)

Fig. 5.5-2 : Effect of bank erosion on the bed slope of bend (Test CCWIG2)

Section	$h_{(b1)}$ (m)	$h_{(b2)}$ (m)	$\Delta h_{(b)}$ (m)	$\frac{\Delta h_b}{h_{(b1)}}$ (%)
3	0.109	0.035	0.074	67.9
4	0.138	0.04	0.098	71.0
5	0.168	0.07	0.098	58.3
6	0.141	0.045	0.096	68.1
7	0.138	0.07	0.061	44.2
Average				61.9

Table 5.5-2: Percentage of the effect of bank erosion on the maximum bend scour depth (Test CCWIG2)

In test CCWIG2, the bank erosion reduced nearly 62 % of bend scour depth near the outer bank, which is shown in Table 5.5-2.

6 DISCUSSION OF RESULTS

6.1 INTRODUCTION

In this chapter the results obtained during the experimental study described in the previous chapter are discussed. The discussions concentrate on the following items:

- experimental facility (Section 6.2);
- accuracy of results (Section 6.3);
- local scour near groynes in straight and in curved channels (Section 6.4);
- combined scour (Section 6.5);
- influence of bank erosion products (Section 6.6);
- recommendations for predictions of combined scour (Section 6.7), and
- recommendations for additional studies (Section 6.8).

6.2 EXPERIMENTAL FACILITY

The experimental non-tilting facility which was designed, constructed and applied for the present study, has some advantages and disadvantages which are discussed hereafter. The main reasons for discussing these are to indicate possible sources of error and to identify improvements that could be made during any subsequent experiments.

The facility was designed to be a simple one, hence it could be built easily in a short time. The height of the facility was sufficient to provide the maximum depth of the scour holes

around the groynes and of the bend scour, as determined by the applied discharges. The flow and sediment distribution were fairly uniformly distributed over the width of the entrance of the channel and all applied instruments had sufficient accuracy for the measurements. The disadvantages of the facility can be listed as follows :

- (1) The width and the length of the facility could have been larger especially for the tests of bank erosion effect (CCWIG) and straight channel without groynes (SCWG). The main problem experienced during the study was the deviation of the final slope from the initial one. This was due to incorrect tailgate position, being based as an estimated value of the hydraulic roughness and its development in time. In subsequent experiments an automatic tailgate positioning system, keeping the slope constant, is recommendable.
- (2) The range of tailgate positions was not ideal for all tests. Therefore, in some cases (such as test SCWG3) the downstream bed level of the channel came under the lowest tailgate level and the tailgate was not too effective any more in regulating downstream water level.
- (3) Although the accuracy of the instruments was very good, it was time consuming to take every sounding of the bed and water level or velocity measurements for all cross-sections. During the required time the bed topography and other parameters were probably changing which may have influenced the accuracy of the measurements.
- (4) The experiments were carried out from March 01, through middle of May 1992. During this period of time, the air temperature increased considerably from first of May. Considering the results of last experiments carried out in May (tests : SCWG2, SCIG2, SCWG3 AND SCIG3), it appears that the increase of air temperature has a significant effect on the water temperature and consequently on the roughness of channels which increased considerably (see also Table 4.3-5). This is commonly experienced for the fine sand used here as bed material. During future experiments the water temperature should be measured and preferably be controlled.

6.3 ACCURACY OF RESULTS

The accuracy of the results is generally found to be affected by various factors like instrumental errors, man made errors, the number of measurements, influence of the number of experiments and the accuracy of the imposed variables. These are discussed hereafter in some detail:

(1) Influence of instrumental errors

The type and the accuracy of the instruments used for measurements can influence the accuracy of the results. In this study however, it is felt that the effect of this factor is fairly small compared to other effects. An exception are the tests on the effect of bank erosion products on a bend scour, where the vertical photographs were the main "measurements" from which the bank erosion rate was determined. This was not very accurate.

(2) Influence of man made errors

This factor is important in the various steps of activities like computations, adjusting the model, using the instruments, reading and writing the data and so on. Cross checking, using the computer, increasing the number of measurements and taking the average value of the various parameters can increase the accuracy. All these were done in the present study.

(3) Number of measurements

The influence of instrumental and man-made errors, but also of any fluctuations of the equilibrium conditions (e.g. due to bedforms cutting into the scour holes) can be reduced by increasing the number of measurements and by spreading them over a sufficiently longer period (several times the period of the fluctuation). No separate study was done into this aspect, but generally speaking three full series of measurement were done once equilibrium had been established. These measurements were spread over a period of two days.

(4) Influence of the number of experiments

It is obvious, that whatever the increase of the number of experiments, the available data will be more and the accuracy will increase. Unfortunately, in this study due to the tight time schedule, only three series of tests were performed including 17 tests, and surely this number was too small for obtaining the perfect results. However, it should be mentioned that the results of this study be received as preliminary results and it is necessary that they have to be completed.

(5) Imposed variables

In addition to the "normal" imposed variables (Q , D_{50} , gradation, width of channel, and slope (or sediment transport, see Chapter 3), there was one parameter that was not fully controlled, notably the water temperature. It is felt that this has caused some problems and it is the main reason not to use the results of the 3rd series.

One way of assessing the accuracy of the studies is to study the reproducibility of the experiments. This can be done comparing experiments with the same boundary conditions. Some idea regarding the accuracy can be obtained by comparing some tests from the 2nd and 3rd series, where the 3rd series then can be used for a check on the accuracy. The two series are only different in slope whereas the other imposed variables remaining the same (possibly apart from the water temperature). In Table 6.3-1 and 6.3-2 a comparison is made between the tests CCWG2 and CCWG3, and GCIG2 and GCIG3, respectively.

Test Number	Slope i (f)	Water Depth h (ave)	Water Velocity v (ave)	Scour Depth h (max) in Sections				
				3	4	5	6	7
	(-)	(m)	(m/s)	(m)	(m)	(m)	(m)	(m)
CCWG2	0.00345	0.076	0.496	0.093	0.163	0.225	0.136	0.129
CCWG3	0.00305	0.094	0.440	0.108	0.158	0.114	0.149	0.225

Table 6.3-1: Comparisons between tests CCWG2 AND CCWG3

Test Number	Slope i (f)	Water Depth h (ave)	Water Velocity v (ave)	Scour Depth h (max) in Sections				
				3	4	5	6	7
	(-)	(m)	(m/s)	(m)	(m)	(m)	(m)	(m)
CCIG2	0.00367	0.091	0.415	0.141	0.246	0.305	0.270	0.268
CCIG3	0.00463	0.082	0.498	0.115	0.229	0.253	0.272	0.258

Table 6.3-2: Comparison between test CCIG2 AND CCIG3

The data in these tables were copied from Table 4.4-1. It can be observed that the slopes are fairly comparable. The observed maximum scour depths are also fairly comparable, apart from sections 5, 6 and 7 in Table 6.3-1.

The conclusion from these comparisons is that the accuracy of the results appears to be in the order of a few centimeters, but some discrepancies exist between the tests CCWG2 and CCWG3.

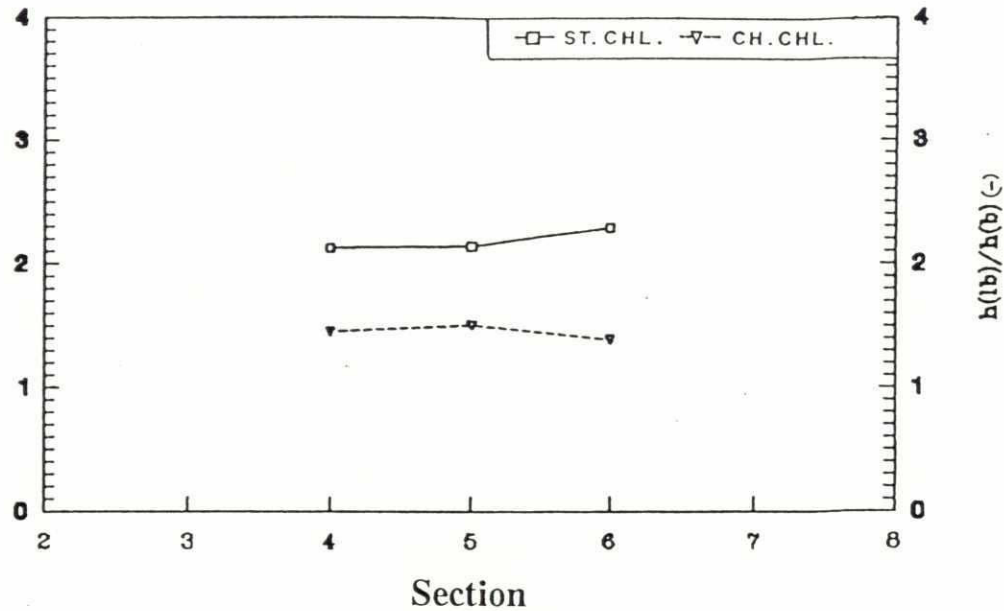
6.4 LOCAL SCOUR AREA NEAR GROYNES IN STRAIGHT AND IN CURVED CHANNELS

The effect of groynes constructed along straight channels and bends was discussed theoretically in Sections 2.3 and 2.4. From an experimental point of view it was discussed in Sections 5.3 and 5.4 based on the results of the tests obtained in the present study. It is clear that structures such as groynes have significant influence on the bed topography around the groyne (erosion) and the pocket between the groynes (deposition).

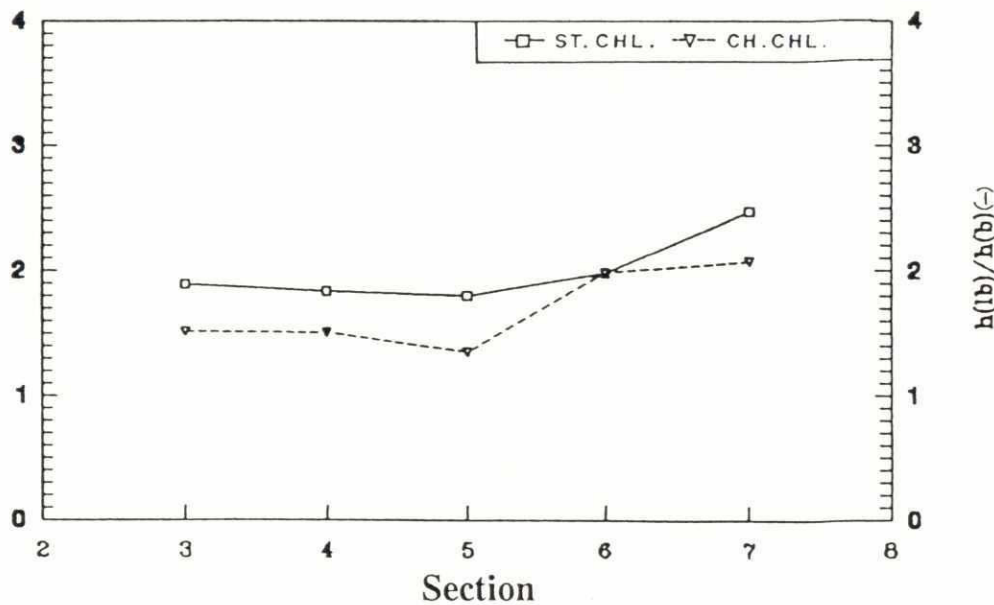
There are several predictors available introduced by various researchers for estimation of the maximum scour depth near groynes in straight channels, but only a few for curved channels. This study is another effort to find a relation which can define the maximum water depth near groynes constructed along the outer bank of bends.

In this section, the local scour due to groynes and the relative importance for both straight and curved channels is discussed. For this purpose, Fig. 6.4-1a and 6.4-1b should be considered, prepared on the basis of the data of the first and second series of the tests. In these figures the maximum local scour depth is made dimensionless by dividing it by the local water depth (without groynes) for both a straight and a curved channel. The figures demonstrate that the effect of groynes constructed in a straight channel is relatively stronger than in a curved channel, because in both series the ratio h_i/h_o is greater than h_i/h_o along the channel.

This is probably due to effect of spiral motion in bends on the created vortices around the head of groynes. Another reason could be the deeper scour hole around the groynes in the curved channel, making the slope of the transverse bed steeper and causing more gravitational force on the bed material forcing them towards the scour hole. If the latter would be dominant, then scale affects would have been present. More studies are needed to elucidate this.



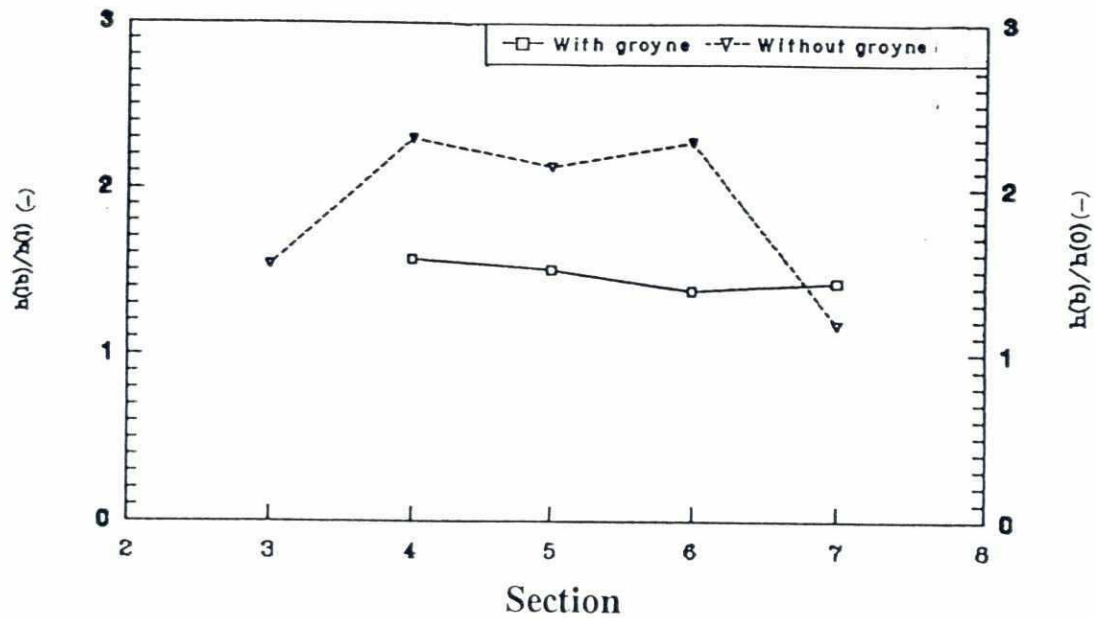
a. first series



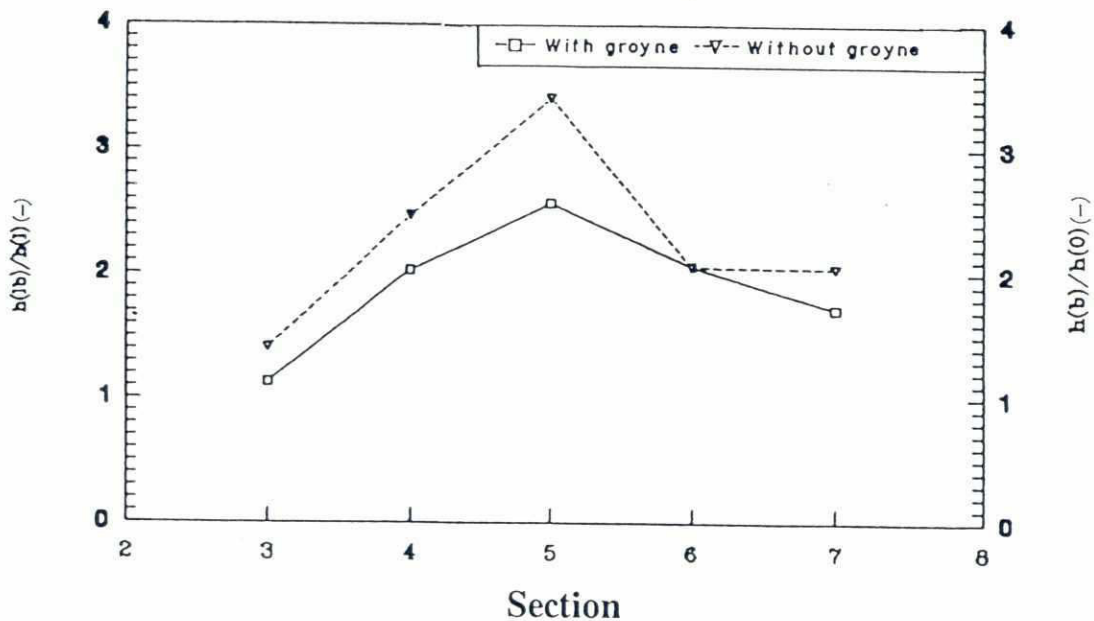
b. second series

Fig. 6.4-1: Groyne effect in straight and curved channels

Something similar holds for the effect of bend flow on the scouring phenomena in the channels with and without groynes. For this purpose, a comparison is made between two ratios, h_b/h_o and h_{lb}/h_l , in the first and second series. Fig. 6.4-2 presents this comparison, which shows that the bend flow is more effective on the relative maximum scour depth in the channels *without* groyne than the channels *with* groynes.



a. first series



b. second series

Fig. 6.4-2: Bend flow effect in channels with and without groynes

6.5 COMBINED SCOUR

In Section 5.4 the different tests were analyzed, and tentatively the following design formula for combined scour (for the combination of local scour and bend scour) was derived:

$$h_b = h_o + h_l + h_b \quad (6.5-1)$$

As discussed in Section 6.3, the inaccuracy of the measurements of the scour holes near the groynes is in the order of a few centimeters. Taking this into account the developed design formula was re-evaluated, considering also the possibilities that observed scour depth in the sections 5, 6 and 7 during test CCWG2 could have been slightly different. It was found that still the tentative design rule remained to be valid. Hence for the time being this design rule was accepted for use.

More comparable series of tests are needed under different conditions (different R/B, B/h and θ) to test the general validity of the above relationship. For translating the result to field conditions, the above design rule should be used rather than scaling up the results obtained during the test using certain scale factors. Especially the local scour depths are not reproduced as scale. For recommendations on the application of the above design rule, see Section 6.7.

6.6 INFLUENCE OF BANK EROSION PRODUCTS

As shown in Section 5.5, the input of bank erosion products has a significant influence on the reduction of the transverse bed slope. Although the obtained data from the related tests (fifth stage of each series) are not sufficient to discuss them quantitatively, it is possible to present some qualitative results as discussed in Section 5.5.

Mosselman (1989) introduced equation (6.6-1) to find the effect of bank erosion products on the transverse bed slope of bends, as follows:

$$\pi \frac{h'}{B_o} = (\theta_o) \left(A \frac{h_o}{R_c} - \frac{1}{\pi} \frac{\eta_o h_o}{S_{xo}} \frac{\delta n}{\delta t} \right) \quad (6.6-1)$$

From this equation it is clear that in addition to the secondary flow effect, A, and transverse bed slope effect, $f(\theta_o)$, also the width of channel (B_o), radius of curvature (R_c) and rate of bank erosion, $\delta h/\delta t$ are important for the transverse bed slope h'/B_o under the condition when significant bank erosion exists. In the present tests not only reduction of the transverse bed slope is due to the input of the bank erosion products, but it is also probably affected by the widening of the channel or even the changing of the radius of curvature of the channel, particularly so, because the direction of the flow was changed towards the outer bend, see Photo 12 to 15.

However, when there is no groyne effect (Fig. 5.3-5), the rising of the bed due to the input of bank erosion products is considerable (see also Section 5.5). In the case of rapid bank erosion, this effect should be considered in designing the protection structures (such as groynes and other revetments), because after protection due to the increased scour, the water depth in the outer bend will increase and will endanger the stability of the structures.

6.7 RECOMMENDATIONS FOR PREDICTION OF COMBINED SCOUR

In view of the contents of the previous Chapters and based on the results of the experiments carried out under this study, the following recommendations are given for the prediction of maximum combined scour depth in front of groynes constructed along the outer bank of bends:

1. Data collection from straight and curved reaches in rivers is advised as first step. These data should include average water depth, discharge, velocity, slope, maximum bend scour, rate of bank erosion and sediment transport rate during the floods, and during average discharge.
2. Prediction of maximum local scour depth at the head of groynes in straight reaches by using predictors like those checked in this study (see Section 5.3). It is advisable to consider several predictions like the ones proposed by Garde and Gill (see also Fig. 5.3-5) and to select the best one based on engineering judgement. If approximate, field measurements in the same or similar rivers should be used.
3. Prediction of axisymmetric bend scour depth and overshoot phenomena for various discharges (particularly for flood condition) by using different predictors. These predictions should be compared with actually measured bend profiles.
4. If determination of rate of bank erosion in bends is possible (for instance, by using aerial photographs or with satellite images), the effect of bank erosion product on the measured profiles should be assessed (see Section 6.6). If important, the influence of blocking this input should be included in the prediction of the bend scour using the formula introduced by Mosselman (1992), as discussed in the previous section and also in Chapter 2.
5. Finally, a combination of average water depth in straight reaches, maximum local scour depth around the groynes in straight channel and maximum bend scour depth can be done by using the design rule resulting from this study (see Section 5.4) as a preliminary prediction.

If and when appropriate physical model tests can be carried out in addition, the possibility of scale effects should be assessed carefully (especially when a movable bed model is used to determine the local scour).

6.8 RECOMMENDATION FOR ADDITIONAL STUDIES

In view of the fact that in this study only three series of tests were conducted and also no other data were available from previous studies, the results of this study should be considered as preliminary. To complete the results, it is necessary to conduct more studies in the future, based on the experiences obtained in this effort. The following studies can be recommended:

- (1) Continuation of the same procedure as used in this study to some improvements on the facility, to use a tilting facility with the approximate dimensions 7×12 m or a non-tilting facility but with the bigger dimensions for reproduction of straight channel and bend simultaneously, see Fig. 6.8-1. The straight and curved reaches of the channel should be long enough to allow for accurate determination of the water level slope. Also the other recommendations listed in Section 6.2 should be taken into account. During this study different combination of R/B , B/h and θ should be studied, possibly also using light weight material.

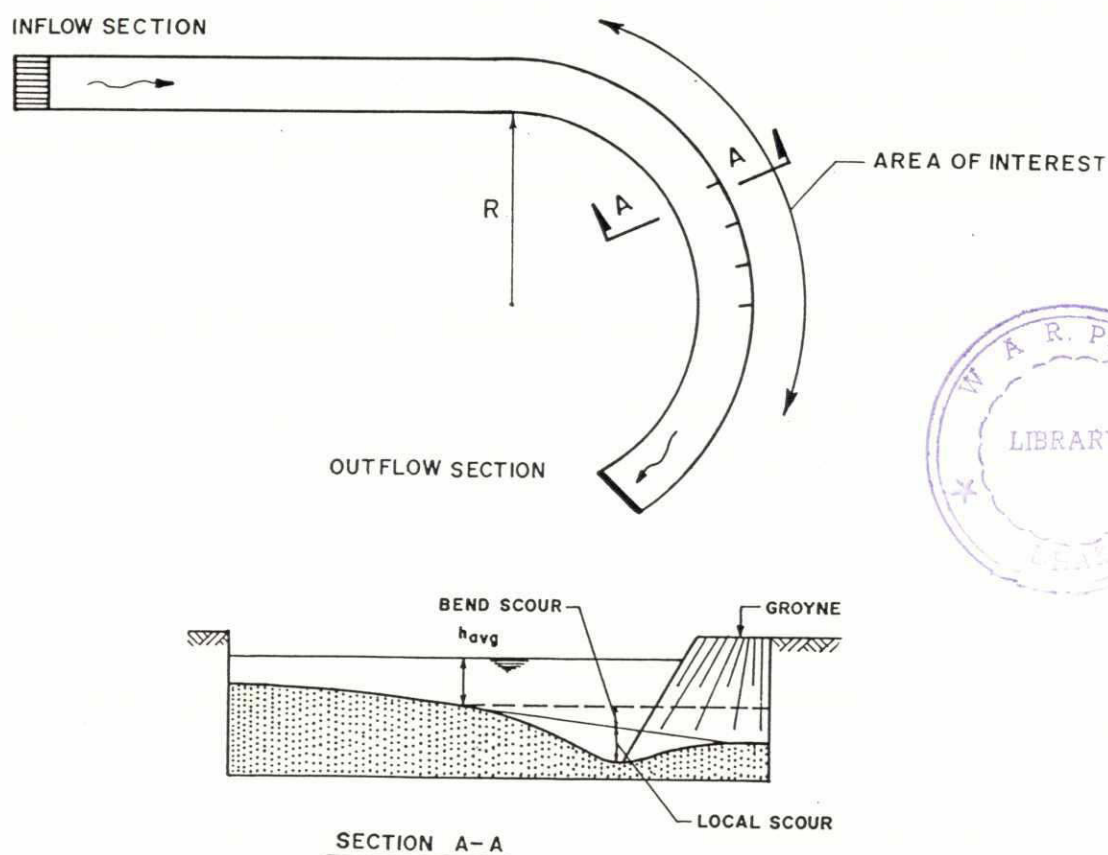


Fig. 6.8-1: Schematized feature of the proposed facility

2. In additions field data should be collected to study the combination of bend scour and local scour. Of course to have the same hydraulic conditions (discharge, slope, Shields parameter and so on) in the straight and the curved channel reach is important.
3. The influence of bank erosion products on the transverse bed slope in bends, which is important for the design of protection structures, needs more experimental or mathematical studies. The experimental studies should use an improved method which does neither affect the groyne nor the width of the channel. Also it is necessary to slow down the bank erosion rate in experimental tests, to have sufficient time to do the various measurements.

7 CONCLUSIONS AND RECOMMENDATION

7.1 CONCLUSIONS

Based on the experimental study described here, the analysis of the results and the subsequent discussions, the following conclusions are arrived at:

1. The approach, as applied here to study the combined scour was found suitable, practical, fruitfull, and yielded interesting and useful results.
2. The water depth near the head of groynes constructed along outer bend can tentatively be determined with the following design rule:

$$h_b = h_o + \Delta h_l + \Delta h_b \quad (7.1-1)$$

where Δh_l and Δh_b are assessed independently (see Section 5.4 and Section 6.7 for details).

3. Under the same hydraulic conditions groynes create higher relative local scour in straight channels than in curved channels, which can be stated as:

$$\frac{h_l}{h_o} > \frac{h_b}{h_b}$$

4. Input of bank erosion products in curved channels has a significant effect in reducing the transverse bed slopes and rising bed level. Hence in designing groynes this effect should be considered (if bank erosion rates before construction the bank protection works found to be considerable).
5. Spacing between groynes has a significant influence on the local scour at the head of groynes in straight channels.

$$h_i = f(D/\lambda_w \text{ or } D/B) \quad (7.1-2)$$

7.2 RECOMMENDATIONS

Further studies are required in order to check the results obtained in this study and to understand the complexities of flow and sediment transport due to construction of groynes along an outer bend. The following recommendations are made for further studies:

1. Continuation of the experiments (if possible using also light weight material) in an improved and larger facility as discussed in Section 6.8 using the same approach as applied the present study aiming at testing the desired design rule for different values for R/B , B/h and θ .
2. Field measurements of local scour near groynes in curved channels should be collected or should be done, and the results should be compared with measurements of bend scour and local scour in straight reaches, to list the design rule developed here.
3. More research on the effect of bank erosion products on the transverse bed slope of bends should be done both theoretically and experimentally.

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PHOTOS



Photo - 1: After the test (scour holes)

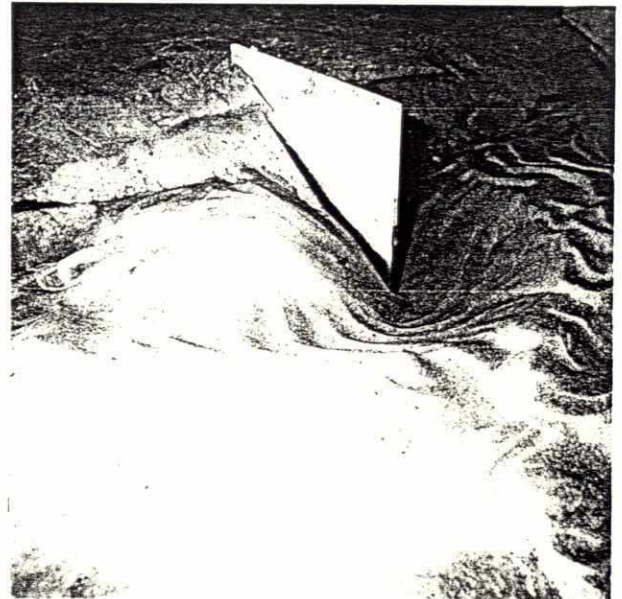


Photo - 2: A feature of scour hole around the groyne

Test : Straight channel including groynes (SCIG1)

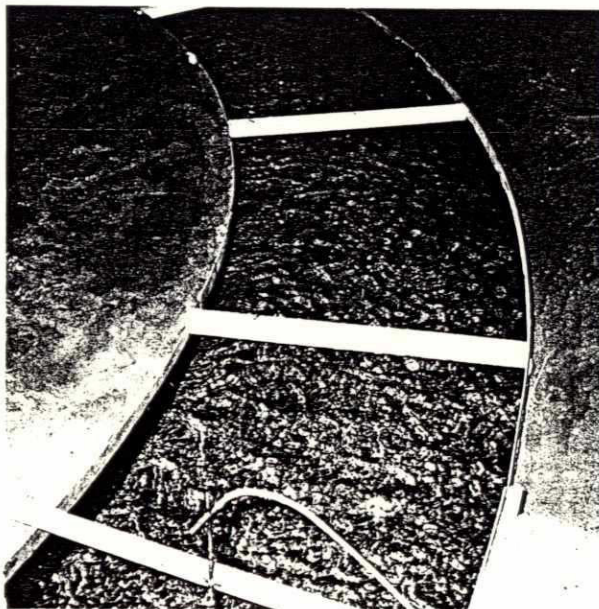


Photo - 3: Test during running

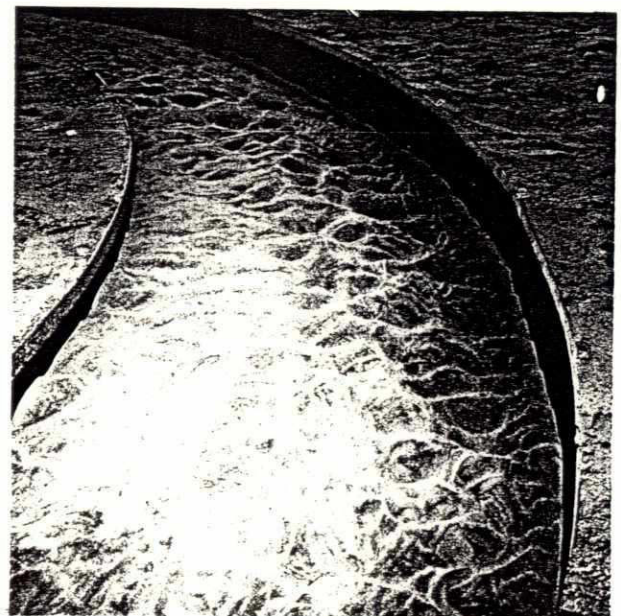


Photo - 4: Bed configuration after running the test

Test : Curves channel without groynes (CCWG1)



Photo - 5: Test after running



Photo - 6: Scour holes around the groynes

Test : Curves channel with groynes along outer bank (CCIG1)



Photo - 7: Bank erosion near groynes caused by the eddies

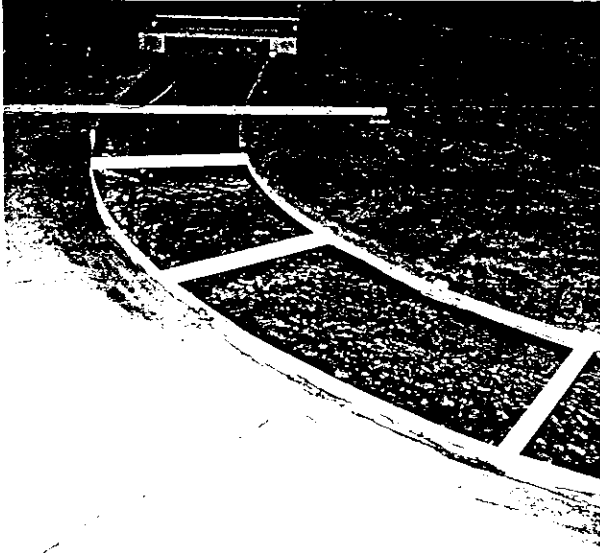


Photo - 8: Test is running during first part



Photo - 9: Test before removing the fixed plates at the outer bend



Photo - 10: Test is running during second part

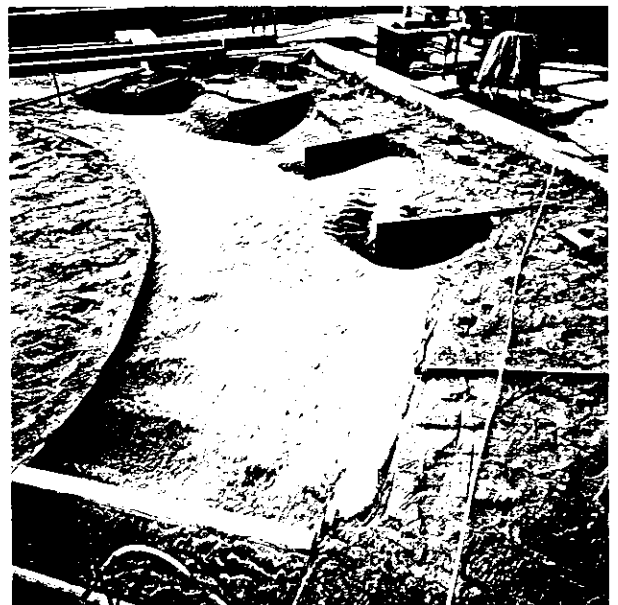


Photo - 11: Feature of the channel after the test

Test : Effect of bank erosion products on bend scour (CCWIG1)

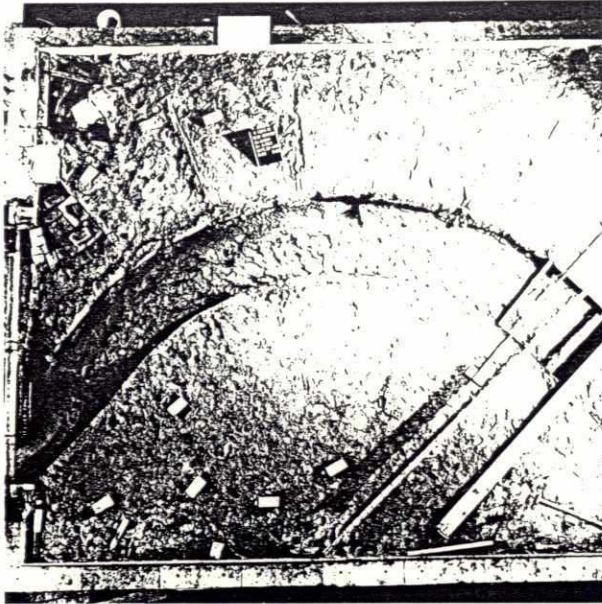


Photo - 12: Just after removing the plates

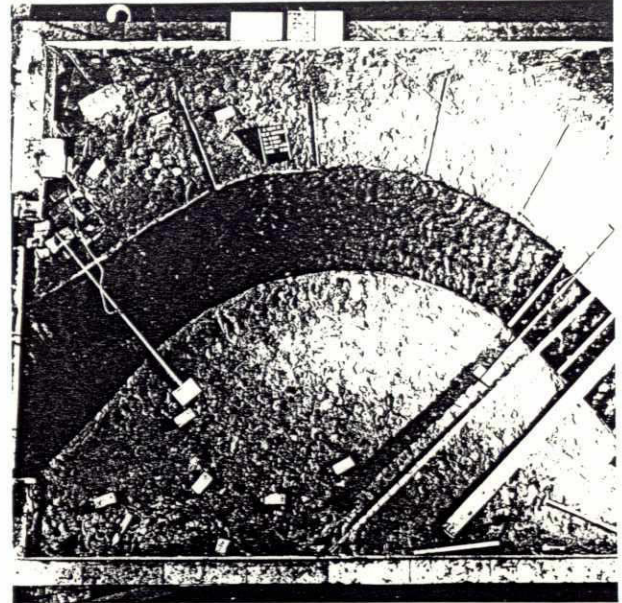


Photo - 13: 25 minutes after running

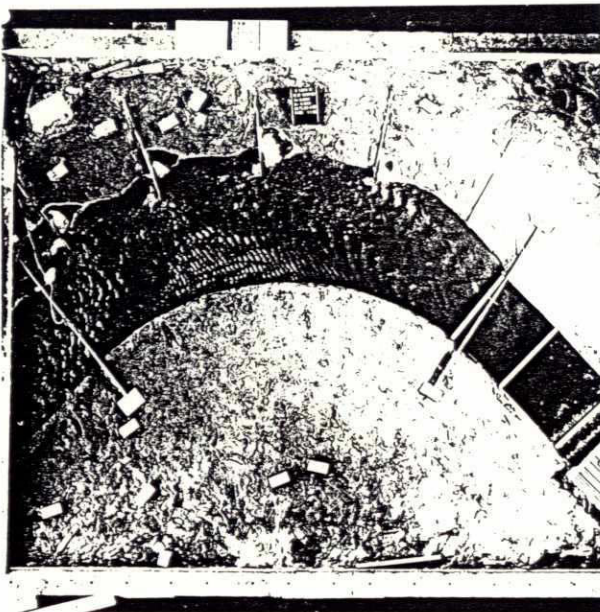


Photo - 14: 2 hours after running

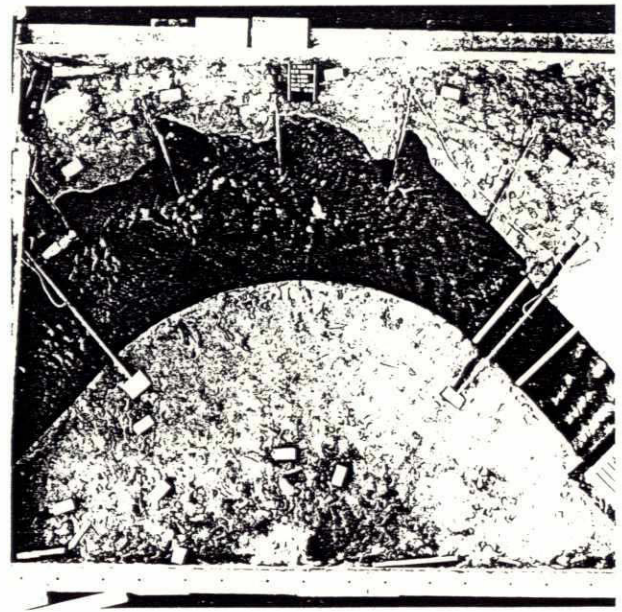


Photo - 15: 4.75 hours after beginning

Test : Bank erosion and widening of the channel

**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 16

DESIGN OF COVER LAYERS OF REVETMENTS

JANUARY 1993

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1 GENERAL

The materials most commonly used for the cover layer of slope revetments may be grouped into different categories (see Subsection 5.1.1 of the Main Report):

- natural material: grass, reed, willows, brushwood, timber work, sand, clay;
- loose units: gravel, rip-rap, concrete blocks;
- interlocking units: concrete blocks of interlocking geometric shapes, cable connected blocks, blocks connected to geotextiles;
- bound or grouted stone: stone layers bound with cement grout or bitumen;
- bituminous systems: asphalt concrete, asphalt mastic, open stone asphalt, dense stone asphalt, lean sand asphalt;
- fabric mattresses and tubes filled with sand, gravel, concrete;
- gabions and mattresses filled with boulders, stones, bricks;
- concrete slabs, and
- asphalt slabs.

The resistance of the protection against hydraulic and other loads is derived from friction, cohesion, weight of the units, friction between the units, interlocking and mechanical strength. However, it is to be stressed that the stability of the cover layer strongly depends on the type and composition of the sublayer. They must be regarded as a whole. Instability of the sublayer and/or the subsoil due to erosion can lead to failure of the cover layer. Therefore cover layer and sublayer are to be designed with an equal probability of failure.

Special attention should be paid to the permeability of cover layer and sublayer. In case the cover layer is designed as impermeable and the river water level falls below the groundwater level, the revetment will be subject to uplift pressures which are to be absorbed by the weight of the revetment resulting in an increase of thickness of revetment layers. Usually impermeable revetments are not required for bank protection of rivers, but for the banks of canals built on embankments above terrain only. Thus the design of impermeable revetments is not followed up in this study.

In the following sections rules for dimensioning different types of cover layers under hydraulic loads being currents and waves are compiled and discussed under consideration of regional experiences where possible and the experience of other FAP-projects.

2 STABILITY OF REVETMENTS UNDER CURRENT ATTACK

2.1 INTRODUCTION

The design of most of the above listed permeable revetments is still more based on rather vague experience than on generally valid calculation methods. Therefore extensive small- and large-scale tests were performed in the Netherlands. Based on the results of these investigations Pilarczyk (1990) developed the following general formula for the calculation of the nominal thickness of a stable cover layer with respect to the current action:

$$D_n = \frac{0.035u^2}{\Delta m \cdot 2g} * \frac{\phi K_r K_h}{K_s \psi_{cr}} \quad (2-1)$$

where D_n	=	nominal thickness of protection unit	(m)
Δm	=	$(\rho_s - \rho_w)/\rho_w$ = relative density	(-)
ρ_s	=	density of protection material	(kg/m ³)
ρ_w	=	density of water	(kg/m ³)
u	=	mean velocity	(m/s)
g	=	acceleration due to gravity	(m/s ²)
ϕ	=	stability factor	(-)
K_r	=	turbulence factor	(-)
K_h	=	depth factor	(-)
K_s	=	slope factor	(-)
ψ_{cr}	=	critical shear stress parameter	(-)

The factors combined in the second term of equation (2-1) take into account the local conditions and the differences between the various materials for cover layers.

The different turbulence factors can be estimated with:

K_r	=	1.0 (normal turbulence in rivers)
K_r	=	1.5 (non-uniform flow with increased turbulence, at outer bends)
K_r	=	2.0 (high turbulence, local disturbances, sharp outer bends)

The following values of the stability factor are recommended, which are strongly affected by of the protection system and its execution:

ϕ	=	0.50 for continuous protection with systems of higher integrity such as gabions, stone mattresses, grouted blocks etc.
ϕ	=	0.75 for continuous protection with loose units
ϕ	=	1.0 for exposed edges of gabions, mattresses, connected blocks etc.
ϕ	=	1.5 for exposed edges of loose units and transition zones.

The slope factor can be obtained from

$$K_s = \cos \alpha \sqrt{\frac{1 - \tan^2 \alpha}{\tan^2 \varepsilon_s}} = \sqrt{1 - \frac{\sin^2 \alpha}{\sin^2 \varepsilon_s}} \quad (2-2)$$

in which

$$\begin{aligned} \alpha &= \text{angle of slope} && (\text{degree}) \\ \varepsilon_s &= \text{angle of internal friction between cover layer} \\ &\quad \text{and sublayer, also called angle of} \\ &\quad \text{response} && (\text{degree}) \end{aligned}$$

The depth factor is defined as follows:

$$K_h = \frac{2}{\left[\log \left(\frac{6h}{D} \right) \right]^2} \quad (2-3)$$

$$\begin{aligned} \text{where } h &= \text{water depth} && (\text{m}) \\ D &= D_n = \text{nominal thickness of protection unit} && (\text{m}) \end{aligned}$$

The values of critical shear stress parameter Ψ_{cr} depend strongly on the protection material/system and will be given in the relevant sections.

2.2 LOOSE UNITS

2.2.1 RIP-RAP

- (1) The nominal size of rip-rap can be found from the following formula (PIANC, 1987a) which is based on the Isbash formula:

$$D_{n50} \geq \frac{0.7 u_r^2}{\Delta m g k} \quad (2-4)$$

where

$$\begin{aligned} D_{n50} &= \text{nominal diameter of rip-rap} && (\text{m}) \\ u_r &= \text{maximum current velocity} && (\text{m/s}) \end{aligned}$$

$$K = \cos \alpha \sqrt{\frac{1 - \tan^2 \alpha}{\tan^2 \varepsilon_s}} = \text{reduction factor}$$

$$\begin{aligned} \varepsilon_s &= \text{natural angle of repose of rip-rap (40°-45°)} \\ \alpha &= \text{angle of slope} && (\text{degree}) \end{aligned}$$

- (2) The formula of Isbash was also presented in the Phase 2 Study Report, River Training Works for Jamuna Bridge Project, however for horizontal bed protection:

$$D = \frac{0.7u_{cr}^2}{\Delta m 2g} \quad (2-5)$$

where u_{cr} = current velocity which causes damage to the bed protection (m/s)

- (3) Another modified Isbash equation is recommended by Hemphill/Bramley (1989) in the same form as the above equation (2-4), however, taking into account the turbulence of flow:

$$D_{n50} = \frac{Cu_c^2}{\Delta mgk} \quad (2-6)$$

where u_c = local flow velocity (m/s)

C = 0.3 for low turbulence, normal river flow

C = 0.7 for high turbulence, including return currents

- (4) The general formula (2-1) of Pilarczyk (1990) can also be used for the calculation of the nominal diameter of rip-rap, assuming the critical shear stress parameter $\Psi_{cr} = 0.035$ for loose material. Since the product of turbulence factor K_t in the range of 1.0 to 1.5 and the stability factor $\phi = 0.75$ for loose units comes approximately to 1.0, equation (2-1) can be simplified:

$$D_n = \frac{u^2}{\Delta m 2g} * K_h * K_s \quad (2-7)$$

where u = mean velocity (m/s)

Δm = relative density (-)

K_s = slope factor, see equation (2-2) (-)

K_h = depth factor, see equation (2-3) (-)

The angle of internal friction between rip-rap and the sublayer needed in equation (2-2) can be assumed:

ε = 20° in case of geotextile

ε = 25° in case of sand

The above equation (2-7) was also proposed in the Phase 2 Study Report, River Training Works for the Jamuna Bridge Project, however without the slope factor K_s , because of applying to a horizontal bed protection.

The same formula was also presented in the Interim Report I of the Brahmaputra River Training Studies of April 1991.

2.2.2 Concrete Blocks

- (1) In PIANC (1987a) the behaviour of concrete blocks under current attack is not considered. However, one formula is given by FAP 1 for concrete cubes in the Design and Construction Management Report of July 1992:

$$D = 0.026 u^2 * \frac{\pi \rho_s - 1}{6 \rho_c - 1} \quad (2-8)$$

where D = diameter of cube (m)
 ρ_s = specific gravity of stone (kg/m³)
 ρ_c = specific gravity of concrete (kg/m³)

- (2) Moreover, a slightly modified equation (2-1) is applicable for the dimensioning of block size :

$$D \geq \frac{\phi \cdot u^2}{\Delta m \ 2g} * K_h * K_s * K_r \quad (2-9)$$

where ϕ = 0,8 for continuous protection
 ϕ = 1,0 for exposed edges
 K_r = 1.0 for normal turbulence
 K_r = 1.5 to 2.0 for high turbulence

The above equation was used for the design of revetments by the Brahmaputra River Training Studies, however, with a value $\phi = 0.7$ and $K_r = 1.0$, see Report on Priority Works, October 1991.

2.3 INTERLOCKING UNITS

Calculation methods for concrete blocks of interlocking geometric shape, cable connected blocks and blocks connected to geotextiles exposed to current attack are not considered in the literature. Only the general formula (2-1) of Pilarczyk can be applied. The relative density Δm of such systems of concrete blocks is in the range of 1.2 to 1.9 and as a mean value $\Delta m \approx 1.4$ may be assumed. The critical shear stress parameter Ψ_{cr} is recommended by Pilarczyk to 0.07, whereas the other factors are to be calculated in accordance with the prevailing conditions in each case.

2.4 GROUTED STONES AND OPEN STONE ASPHALT

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

2.5 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).

$$d_{n50} = h * \left[\frac{u_{cr}}{B \sqrt{k' \psi_{cr} g \Delta m h}} \right]^{2.5} \quad (2-10)$$

where

d_{n50} = grain diameter (m)

$$= \left[\frac{W_{50}}{\rho_s} \right]^{\frac{1}{3}}$$

h = water depth (m)

K' = slope reduction factor (-)

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

$$= \sqrt{1 - \frac{\sin^2 \alpha}{\sin^2 \varepsilon_s}}$$

ε_s = angle of internal friction of material

Ψ_{cr} = Shields parameter

= 0.1 in case of initiation of movement

B_1 = coefficient for flow conditions

= 5 to 6 for major turbulent flow and outer bends of rivers

= 7 to 8 for normal turbulence of rivers

= 8 to 10 for uniform flow and minor turbulence

Equation (2-10) was also discussed and used for River Training Works for Jamuna Bridge Project, see Study Report, Phase 2 of May 1990.

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.

$$t_m \approx 1.8 d_{m50} \quad (2-11)$$

in which

t_m = thickness of mattress (m)

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

$$D_n = \frac{\bar{u}^2}{\Delta m \ 2g} * \frac{0.03}{\psi_{cr}} * K_h * K_s \quad (2-12)$$

where $\bar{u}$	=	current velocity	(m/s)
Ψ_{cr}	=	Shields parameter	(-)
K_s	=	slope factor, see equation (2-2)	(-)
K_h	=	depth factor, see equation (2-3)	(-)

Since the stones are retained by the wire mesh, the Shields parameter can be increased from $\Psi_{cr} = 0.03$ to $\Psi_{cr} = 0.06$ to 0.1 in case of initiation of movement.

The above formula of Pilarczyk was also discussed and slightly modified for the prevailing conditions in the Phase 2 Study Report, River Training Works for Jamuna Bridge Project. Moreover, use was made also of the above equation for the Meghna River Bank Protection, see Final Report of Short Term Study, March 1990.

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

$$d \geq \frac{\phi \bar{u}^2}{\Delta m 2g} * K_h * K_s \quad (2-13)$$

where d	=	thickness of mattress	(m)
ϕ	=	0.8 for continuous protection	(-)
ϕ	=	1.0 for exposed edges	(-)
K_s	=	slope factor, see equation (2-2)	(-)
K_h	=	depth factor, see equation (2-3)	(-)

3 STABILITY OF REVETMENTS UNDER WAVE ATTACK

3.1 INTRODUCTION

Wave attack to revetments may be originated from wind waves or by ship induced waves. The latter are generally the main design criteria for rivers and inland navigation channels plied by large vessels. For the Jamuna river with its relatively low traffic of generally small ships, the wind waves are more important particularly as the river's widths gives a long fetch length to develop waves. Besides the fetch, the factors affecting the wave action are the wind speed and duration, and the water depth. Brief gusts with high velocities do not last long enough to cause wave growth, and the water depth limits the wave heights both during generation and through the breaking criterion.

Among other equations, a general empirical formula was developed by Pilarczyk (1990) for the calculation of the specific size or the thickness of the protection unit:

$$D \geq \frac{H_s \xi_z^b}{\Delta m \psi_u \phi \cos \alpha} \quad (3-1)$$

where D	=	specific size or thickness of protection unit	(m)
H _s	=	significant wave height	(m)
Δm	=	relative density of a system unit	(-)
φ	=	stability factor	(-)
Ψ _u	=	system determined stability upgrading factor	(-)
	=	1.0 for rip-rap as a reference	
	≥	1.0 for other revetment systems	
α	=	slope angle	(degree)
ξ _z	=	breaker similarity index	(-)

$$= \tan \alpha \left[\frac{H_s}{L_o} \right]^{-0.5} \quad (3-2)$$

$$= \frac{1.25 T_z \tan \alpha}{\sqrt{H_s}} \quad (3-3)$$

where L _o	=	wave length = $\frac{g T_z^2}{2\pi}$	(m)
T _z	=	average wave period	(s)

The exponent b in equation (3-1) related to the interaction process between the waves and the revetment type (roughness, porosity, permeability etc.) is in the range between 0.5 and 1.0. For rough and permeable revetments as rip-rap b = 0.5 can be used. For smooth and less permeable placed - block revetments it can be close to b = 1.0. For other systems such as more open blocks, block mats, mattresses etc. the value b = 2/3 can be treated as a common representative value.

Formula (3-1) is limited by $\cot \alpha > 2.0$

3.2 LOOSE UNITS

3.2.1 Rip-Rap

The nominal size of rip-rap (2 layers) can be determined with the general Pilarczyk formula (3-1) taking into account $\Psi_{cr} = 1.0$ and $b = 0.5$:

$$D_n = \frac{H_s \xi_z^{0.5}}{\Delta m \phi \cos \alpha} \quad (3-4)$$

Pilarczyk recommends to use $\phi = 2.25$ for rock. The same equation (3-4) is also recommended by PIANC (1987a) with ϕ in the range of 2 to 3 on condition that the filter requirements of the soil have been met by a geotextile filter, that the subsoil is at worst sand and that the impinging wave height is not greater than 1.2 m. PIANC (1987a) presents also the following table, from which values of the wave breaking parameter can be taken:

Type of Breaking Wave	Smooth Slope	Slope with rip-rap Protection
Surging		$\xi > 3.0$
Collapsing	$\xi > 3.3$	$2.0 < \xi < 3.0$
Plunging	$0.5 < \xi < 3.3$	$\xi < 2.0$
Spilling	$\xi < 0.5$	

Table 3.2-1: Wave breaking characteristics

(Source: PIANC, 1987a)

3.2.2 Concrete Blocks

- (1) The specific size of loose closed concrete blocks placed either on a granular filter or on a geotextile on sand can be determined by application of the general Pilarczyk formula considering $b = 2/3$ and $\Psi_u = 1.5$:

$$D = \frac{H_s \xi_z^{2/3}}{\Delta m 1.5 * 2.25 \cos \alpha} \quad (3-5)$$

- (2) A modified Pilasczyk formula is again presented by PIANC (1987a):

$$D = \frac{H_s \xi_z^{0.5}}{\Delta m \varphi \cos \alpha} \quad (3-6)$$

Differences of revetment material and local conditions can be taken into account by use of different values of strength coefficient $\varphi = 2.25 \phi$.

For loose concrete blocks φ is to be used in the range of 3 to 4 on condition that the filter requirements of the soil have been met by a geotextile filter, that the subsoil is at worst sand and that the impinging wave height is not greater than 1.2 m. In case loose concrete blocks should be placed on clay, the strength coefficient may range between 5 and 6, however the clay has to meet the following requirements:

- plasticity index $I_p > 20 \%$
- consistency index $I_c > 0.8$
- at least 95 % of Proctor density
- smooth, crack-free and homogeneous surface.

- (3) PIANC (1987a) mentions also the classical Hudson formula, which however, does not take into account the wave period.

$$W = \frac{\rho_c H^3}{K_D \Delta m \cot \alpha} \quad (3-7)$$

where W	=	weight of armour unit	(t)
K_D	=	damage coefficient	(-)
Δm	=	relative block density	(-)
ρ_c	=	density of block unit	(t/m ³)

K_D - values are to be taken from Shore Protection Manual, CERC (1984)

- (4) For concrete blocks under wave attack FAP 1 presented in the Design and Construction Management Report of July 1992 the following equation, also developed by Pilarczyk:

$$D = \frac{H_s}{\rho_s - 1} \cdot \frac{\sqrt{\xi}}{\cos \alpha} \cdot \frac{1}{\beta} \quad (3-8)$$

where β	=	strength coefficient
	=	3 for cubes (placed)
	=	2 for randomly dumped cubes
ρ_s	=	specific gravity of revetment material

3.3 INTERLOCKING UNITS

- (1) When applying the Pilarczyk formula for interlocking blocks or block- mats, exponent b should be 2/3 and the stability factor $\Psi_u = 1.5$ for blocks connected to geotextile placed on a granular sublayer and $\Psi_u = 2.0$ for cable connected blocks, also placed on a granular sublayer; resulting in:

$$D = \frac{H_s \xi_z^{2/3}}{\Delta m \ 1.5 * 2.25 \cos \alpha} \quad (3-9)$$

for blocks connected to geotextile and

$$D = \frac{H_s \xi_z^{2/3}}{\Delta m \ 2.0 * 2.25 \cos \alpha} \quad (3-10)$$

for cable connected blocks

- (2) A modified Pilarczyk formula is again presented by PIANC (1987a), however, with $b = 0.5$ and some recommendations for the so-called strength coefficient $\varphi = 2.25\phi$.

$$D = \frac{H_s \sqrt{\zeta}}{\Delta m \varphi \cos \alpha} \quad (3-11)$$

For blocks interlocked by friction, cable connected blocks and grouted blocks connected to a geotextile φ should be according to the PIANC recommendations in the range 4 to 5 on condition that the filter requirements of the soil have been met by a geotextile filter, that the subsoil is at worst sand and that the impinging wave height is not greater than 1.2 m. If blocks connected to a geotextile should be used without grout, the strength coefficient must be $\varphi = 4$ and it is implied that a sublayer and an additional geotextile will be incorporated.

The same Pilarczyk formula may also be applied for grouted cabled blocks and mechanically interlocked blocks with a strength coefficient $\varphi \geq 6$. However, in such cases large scale model testing is recommended.

3.4 GROUTED STONES

In case of grouted stone used as cover layer on a sloped 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 (see equation 3-1) b in the range of 0.5 to 2/3 and the stability factor.

$$\begin{aligned} \Psi_u &= 1.05 \text{ for surface grouting (30 \% of voids)} \\ \Psi_u &= 1.50 \text{ for pattern grouting (60 \% of voids)} \end{aligned}$$

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

$$D_{n50} \geq \frac{H_s}{\Delta m (K_D \cot \alpha)^{0.33}} \quad (3-12)$$

where K_D = damage coefficient as per CERC (1984)

However, it should be taken into consideration that the above equation is based on the Hudson formula which has the limitation as already pointed out in Subsection 3.2.2. 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

$$\begin{aligned} D_{n50}(\text{grout}) &= 0,9 D_{n50}(\text{loose}) \text{ for surface grouting} \\ D_{m50}(\text{grout}) &= 0,6 D_{n50}(\text{loose}) \text{ for pattern grouting} \end{aligned}$$

3.5 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, see PIANC (1987a):

$$d_b = 0.75 \left[\frac{27}{16} \frac{1}{1-V^2} \left(\frac{P}{\sigma_b} \right)^4 \left(\frac{S}{C} \right) \right]^{\frac{1}{3}} \quad (3-13)$$

where d_b	=	thickness of cover layer	(m)
σ_b	=	asphalt stress at failure	(N/m ²)
P	=	wave impact	(N/m)
S	=	stiffness modulus of the asphalt	(N/m ²)
V	=	Poisson's ratio for asphalt	(-)
C	=	modulus of subgrade reaction	(N/m ³)

The following thicknesses of open stone asphalt layers on river banks can be estimated as a guidance.

small rivers:	d_b	=	10 to 15 cm (insitu)
	d_b	=	8 to 12 cm (prefabricated mattress)
large rivers:	d_b	=	15 to 25 cm (insitu)
	d_b	=	15 cm (prefabricated mattress)

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

$$b = 2/3$$

$$\Psi_u = 2.0 \text{ for open stone asphalt placed on geotextile on sand}$$

$$\Psi_u = 2.5 \text{ for open stone asphalt placed on sand asphalt as sublayer. In this case the thickness of the system may be defined as the total thickness of both layers.}$$

3.6 GABIONS AND MATTRESSES

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 mattress and

$$d \cong 1.8 D_n \quad (3-14)$$

- (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:

$$d = \frac{H_s \xi_z^{0.5}}{\Delta m \psi_u 2.25 \cos \alpha} \quad (3-15)$$

where $\Psi_u = 2 \text{ to } 3$

$\Delta m =$ relative density of the system

$\Delta =$ relative density of fill material $= (1-n) \Delta m$

$n =$ bulk-porosity of fill material

$$D_n = \frac{H_s \xi_z^{0.5}}{\Delta m \psi_u 2.25 \cos \alpha} \quad (3-16)$$

in which

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

$\Delta m \approx 1.65$

The application of equation (3-15) and (3-16) is limited to $H_s < 1.5 \text{ m}$.

- (2) For the estimation of thickness of the gabions/mattresses PIANC (1987a) gives the following equations:

$$t_m = \frac{H_s}{2(1-n)\Delta m \cot \alpha} \quad \text{for } \cot \alpha < 3.0 \quad (3-17)$$

$$t_m = \frac{H_s}{4(1-n)\Delta m (\cot \alpha)^{1/3}} \quad \text{for } \cot \alpha \geq 3.0 \quad (3-18)$$

where n = porosity of stone ≈ 0.4

However, these equations are only used for significant wave heights of less than 1 m.

Since in case of common quarry stone $(1-n)\Delta m \approx 1.0$, the above equations become

$$t_m = \frac{H_s}{2 \cot \alpha} \quad \text{for } \cot \alpha < 3.0 \quad (3-19)$$

$$t_m = \frac{H_s}{4(\cot \alpha)^{1/3}} \quad \text{for } \cot \alpha \geq 3.0 \quad (3-20)$$

3.7 FABRIC CONTAINERS

The stability criterion for fabric mattresses of thickness d filled with sand, sand-cement or other materials depend on the permeabilities of the mattress and the subsoil. This can be taken into account by the stability factor Ψ_u in the Pilarczyk formula, the use of which is limited to $H_s \leq 1.5$ m. The permeability of the mattress should be treated as an integrated permeability of all the components i.e. geotextile container and fill material together. The following values of stability factor may be used:

- for $P_m < 1$: $\Psi_u = 1.0$ (less permeable)
- for $P_m \approx 1$ to 2 : $\Psi_u = 1.5$
- for $P_m > 1.0$: $\Psi_u = 1.75$ to 2.0 (permeable mattress)

Where P_m is the ratio of the permeabilities of the mattress and the subsoil. The exponent b should be about 2/3.

4 GRADING OF LOOSE STONES

Small stones, cobbles and small boulders used as rip-rap are specified by size and grading. The size is usually characterized by the nominal diameter D_{n50} . Larger stones are preferably specified by weight. The relation between nominal size and weight is

$$D_{n50} = \left(\frac{W_{50}}{\rho_s} \right)^{0.33} \quad (4-1)$$

where

ρ_s = density of stone, typically in the range of 2500 to 2700 kg/m³
 W_{50} = 50 % value of the mass distribution curve of the stone

Rip-rap should have a smooth grading curve and be well graded so that material smaller than W_{15} does not exceed the volume of the voids between the larger rocks. Rounded stones are less stable than angular ones. For best stability they should be blockish in shape. However, the maximum dimension is usually limited to about 3.0 times the minimum.

Though there is no standard definition of the grading of stones, many specifications for the material fall within the following upper and lower size limits, recommended by PIANC (1987a):

- the upper limit of the W_{100} stone should equal the maximum size that can be economically obtained from the quarry but not exceed four times w_{50} min;
- the lower limit of the W_{100} stone should not be less than twice W_{50} min;
- the upper limit of the W_{50} stone should be about 1.5 times W_{50} min;
- the lower limit of the W_{15} stone should be about 0.4 times W_{50} min, and
- the upper limit of the W_{15} stone should be slightly greater than W_{50} min

These are illustrated in Fig.4-1

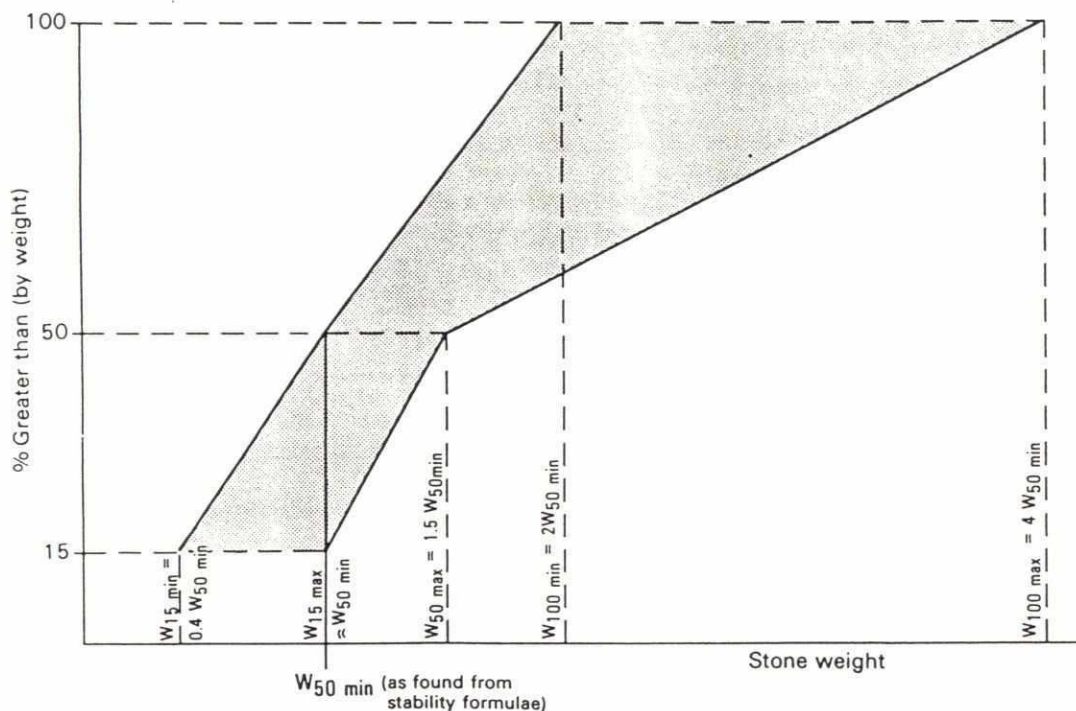


Fig.4-1: Grading envelop for rip-rap

(Source: PIANC, 1987a)

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**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 17

DESIGN OF FILTER LAYERS OF REVETMENTS

JANUARY 1993

DESIGN OF FILTER LAYERS OF REVETMENTS

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1 GENERAL

The stability of the whole revetment depends strongly on the type and composition of the filter layer. The function of a filter is

- to prevent migration of subsoil particles out of the bank slope, and
- to allow at the same time movement of water through the filter.

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

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

- granular filters, made of loose, bounded or packed grains, and
- fibre filters, 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.

Seminar on Filter Technology

Considering the paramount importance of correct filters in bank protection works on the Jamuna and other rivers with similar soil conditions, and bearing in mind the limited know-how available on that field in Bangladesh as well as the controversial discussion on the use of geotextiles, the Consultant arranged a 2 days seminar on filter technology in September 1992. The seminar was attended by participants of FPCO, BWDB, BUET, BJMC (Bangladesh Jute Mills Corporation), BJRI (Bangladesh Jute Research Institute), presenting an attractive mingling of users, manufacturers, researchers and designers. The Consultant took part with his senior professional staff supplemented by two experts. During the discussions it turned out that the seminar was the first forum whatever where endusers (BWDB), manufacturers (BJMC) and researchers (BJRJ, BUET) working in Bangladesh had a joint discussion.

- The term geotextiles defines the difference to granular filters, it does not imply the type of materials. Hence geotextile filters may be made of synthetic or natural fibers.
- The decision on the type of fibers (geosynthetic or geonaturals) depends on the functionality and durability of the designed filter component.
- In all cases where long term durability is of minor importance, geonatural fibers can be used without problems.

- In many cases a composite filter made of geonaturals and geosynthetics promises to be the optimum solution in Bangladesh using the natural fibers to give the required strength for the placement procedures and the synthetic fibers for long term durability of the filter component.

During the Test and Implementation Phase the Consultant intends to continue his activities in applied research on practical aspects of filter technology with a particular stress on the conditions in Bangladesh. He will continue and intensify the contacts with the institutions related to the promotion of jute in Bangladesh (BJMC, BJRI) and he will assist in developing new standards for Bangladesh application.

Filter Test Rig

One of the major tools in that respect will be the filter test rig which would allow investigations on granular and geotextile filters and which due to circumstances not under the Consultant's control could not be erected within the Planning Study Phase. It is now intended to erect the test rig in an existing shed on the BWDB premises in Green Road and to have it run in cooperation (under a subcontract) with AIR (Applied and Innovative Research Ltd.). Close contacts are intended to be held with other researchers in Bangladesh in particular from BUET and BJRI.

2 GRANULAR FILTERS

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

In the past some simple filter rules had been developed which are presented below :

According to the US Bureau of Reclamation :

- for soils with $U < 5$:

$$D_{50} < (5 \text{ to } 10) * d_{50} \quad (2-1)$$

- for soils with $U > 5$:

$$D_{15} < (12 \text{ to } 40) * d_{15} \quad (2-2)$$

and at the same time

$$D_{50} < (12 \text{ to } 58) * d_{50} \quad (2-3)$$

where			
U	=	d_{60}/d_{10} = coefficient of uniformity	(-)
d_x	=	particle size of subsoil from a grain size distribution plot at x % finer by mass	(mm)
D_x	=	particle size of filter material from a grain size distribution plot at x % finer by mass	(mm)

However, it must be observed, that the filter material does not contain sizes > 65 mm and not more than 5 % with sizes < 0.074 mm.

Another rule was given by US Corps of Engineers :

$$D_{15} < 5 d_{85} \quad (2-4)$$

$$D_{15} < 20 d_{15} \quad (2-5)$$

$$D_{50} < 25 d_{50} \quad (2-6)$$

According to Pilarczyk (1990) a granular filter has to meet the following requirements:

$$D_{15} < (4 \text{ to } 5) * d_{85} \quad (\text{stability criterion}) \quad (2-7)$$

$$D_{15} > (4 \text{ to } 5) * d_{15} \quad (\text{permeability criterion}) \quad (2-8)$$

$$D_{50} < (20 \text{ to } 25) * d_{50} \quad (\text{segregation criterion}) \quad (2-9)$$

$$U = \frac{D_{50}}{D_{10}} < 10 \quad (\text{no migration}) \quad (2-10)$$

$$U = \frac{D_{50}}{D_{10}} > 20 \quad (\text{migration}) \quad (2-11)$$

The lower factors in the above equations relate to strong cyclic flow conditions and the higher ones to steady flow conditions. In all cases, the grading curve of the filter should be approximately parallel to that of the subsoil.

3 FIBRE FILTERS

3.1 INTRODUCTION

3.1.1 General

The principal requirements of fibre filters are similar to those of granular filters. The filter must fulfil the retention criterion and the permeability criterion. However, it should be observed that 2 phenomena can reduce the efficiency of the filter. The permeability can considerably decrease when grains from the soil to be protected form a cake on the surface of the filter or when grains are trapped in the fibre pores resulting in clogging of the fibre.

Since fibre filters are mostly referred to as geotextiles, this term will be used in the following. It should be pointed out that the term geotextiles does not refer necessarily to synthetic fibres, because they can - for certain applications - also be produced from natural fibres, as from jute. The Consultant prefers, therefore, to use the terms geonaturals and geosynthetics for identifying the material whereas the term geotextiles is used without identifying the type of material.

The filter performance of geotextile filters depends on the properties of the subsoil and of the geotextile.

Properties of the subsoil :

- characteristic particle size, generally expressed as the sieve size d_x , that x % by mass of the subsoil particles are smaller than that size;
- uniformity $U = d_{60}/d_{10}$;
- state of compaction;
- cohesive or non-cohesive, and
- permeability in the direction of flow.

Properties of the geotextile:

- characteristic opening size, generally expressed as the pore size O_n , that n % of the pores are smaller than that size;
- type of fabric: woven or non-woven (e.g. needle punched);
- permeability, generally expressed in terms of permittivity;
- resistance to stress and puncturing, elongation, and
- durability.

Important for the design of revetments including the filter layer are the hydraulic properties of the latter, notably its permeability and its filtration characteristics.

Geotextiles are usually highly permeable materials. The flow through the geotextile is normally laminar when the geotextile is embedded in the soil but becomes turbulent when subjected to wave action under a permeable cover layer of rip-rap or concrete blocks. It must be taken into account that the water flow through the fabric may be normal to its plane or in its plane.

The permeability of a geotextile normal to its plane is usually measured in the laboratory and can be expressed by the following equation:

$$k_g = \frac{q * t_g}{\Delta H * a_g} \quad (3-1)$$

where k_g	=	permeability of geotextile	(m/s)
q	=	rate of flow	(m ³ /s)
a_g	=	surface area of geotextile	(m ²)
ΔH	=	head loss	(m)
t_g	=	geotextile thickness	(m)

Alternatively the permeability may be expressed as permittivity being the permeability of the geotextile normal to its plane related to its thickness:

$$\psi = \frac{k_g}{t_g} = \frac{q}{\Delta H * a_g} \quad (3-2)$$

where Ψ	=	permittivity	(s ⁻¹)
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Permittivity values of geotextiles vary between 10⁻² and 10 s⁻¹.

In case a geotextile is used as a drain within its plane, the water flow in the plane is expressed as transmissivity. Values of the transmissivity depend on the type and thickness of the product. Flat and woven geotextiles usually have a lower transmissivity than thick and non-wovens.

3.1.2 Permeability Criteria

For the proper and continuous functioning of a filter layer it must be ensured that the filter always maintains a higher permeability than the soil. Taking into account safety factors for the soil heterogeneity, the compressibility of the fabric, for blocking and clogging in the order of 10² to 10³, the following formula is given in PIANC (1987a)

$$\Psi > (10^2 \text{ to } 10^4) * k_s * i \quad (3-3)$$

where k_s	=	permeability of soil	(m/s)
i	=	hydraulic gradient in the soil	

Clogging occurs when soil particles are trapped within the geotextile. Blocking is the accumulation of particles on the upstream face of the geotextile.

An alternative approach using the permeability is given in PIANC (1987a), Appendix B.

3.1.3 Retention Criteria

The geotextile filter must have an opening size small enough to prevent migration of soil particles. The following factors have to be taken into consideration, when assessing the opening size of the geotextile:

- type of soil (cohesive or non-cohesive)
- density of soil (loose or compact)
- grading of soil (uniform or non-uniform)
- magnitude of hydraulic gradient
- type of fabric (woven, non-woven)

The retention criterion can be expressed as

$$O_n < \lambda d_n \quad (3-4)$$

where O_n = is the opening size of the geotextile corresponding to the diameter of the largest particles able to pass through the geotextile. O_n corresponds to the d_n of the soil passing through the geotextile: n is normally taken as 98, 95 or 90

d_n = is chosen as a characteristic diameter of the retained soil. As a representation of the large diameter particles within the soil d_{85} or d_{90} are normally used; for medium size particles d_{50} is used.

λ = is a coefficient depending upon the factors listed above together with the type of test used to determine O_n (dry, wet, wet turbulent or wet alternate flow).

Many different equations have been derived from the above formula (3-4) and there are various methods of determining the size of openings of a geotextile varying from country to country. Therefore, it is essential that the used test is compatible with the adopted design criteria.

3.2 FILTER RULES

Hemphill/Bramley (1989) give a filter rule, where the filter properties are generally specified in terms of fabric type, pore size and permeability. Under steady flow conditions, for soil retention:

$$O_{90} < \lambda \cdot d_x \quad (3-5)$$

where values of λ range from 1.0 to 2.0 depending on soil density and uniformity.

Further criteria for O_{90} are given by Ingold (1984) and Heerten and Wittman (1984), who also recommend that the maximum size should not exceed about 0.3 to 0.5 mm.

As explained above, the design approach must also take into account the reduction in permeability of the as-manufactured fabric due to clogging and/or blocking of the fabric pores in service. Heerten and Wittman (1984) give an empirical method for assessing the permeability reduction factor η due to clogging/blocking which is applied to the as-manufactured permeability k_g :

- for non-woven fabrics, needle punched and thicker than 2 mm: $\eta = 0.2$
- for woven fabrics the reduction factor is itself dependent on as-manufactured permeability and soil particle size d_{10} , as shown in Fig.3.2-1.

This criterion may also be applied to thin non-woven fabrics.

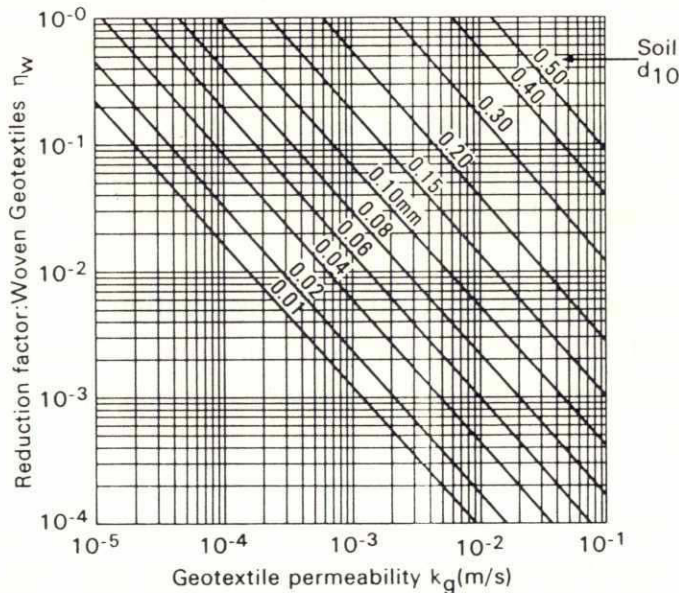


Fig.3.2-1: Permeability reduction factor for woven geotextile fabrics

(Source: Heerten and Wittmann, 1984)

After the reduction factor has been estimated, the permeability of the geotextile in service can be compared with that of the subsoil.

3.3 DESIGN PROCEDURE AS PER PIANC

PIANC (1987a) provide very general criteria for retention and permeability as a first estimate

(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 greater than 5:
 $0.05 \text{ mm} < O_{90} < d_{90}$

(b) Permeability

$$\psi > 5 \times 10^3 * k_s * i \quad (3-6)$$

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

However, it is also mentioned that additionally the susceptibility of the soil to down slope migration must be assessed. Soils with a tendency to down slope migration are silts, sandy silts and fine sand. In such cases steps must be taken to control the downslope migration.

PIANC (1987a) presents in Appendix B one particular design method which is based on experience gained in Germany and was developed by Heerten and Wittmann (1984). The design procedure is illustrated in Fig.3.3-1.

(1) Determination of the Range of the Grain-size Distribution

The soil retention criteria for the filter are dependent upon the grain-size distribution of the soil. For this reason the soil distribution curve is classified as range A, B or C in accordance with the following rules,

- Range A : 40 % or more of the soil particles will be smaller than or equal to 0.06 mm.
 Range B: 15 % or less of the soil particles will be smaller than or equal to 0.06 mm.
 Range C: Between 15 % and 40 % of the soil particles will be smaller than or equal to 0.06 mm.

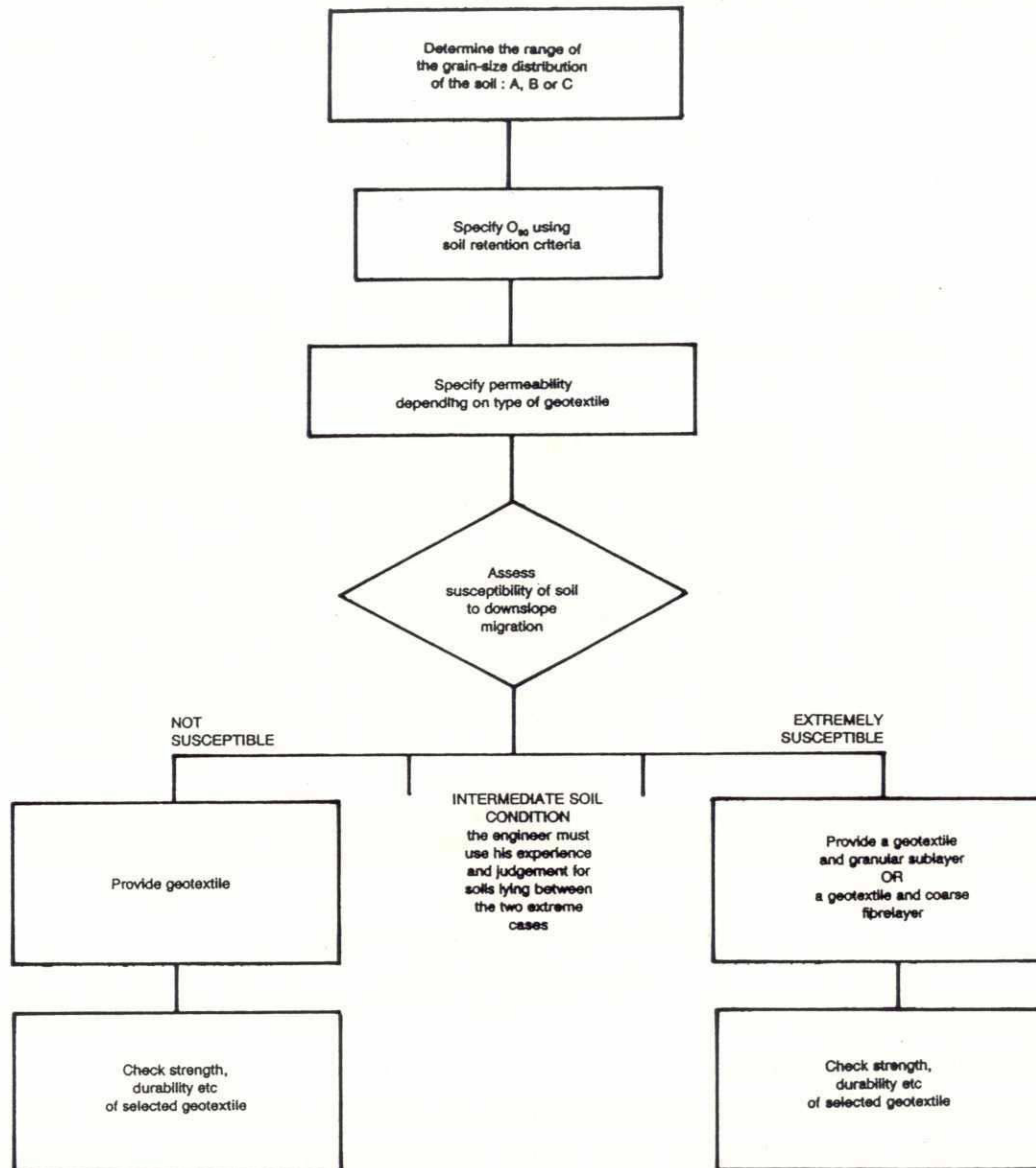


Fig.3.3-1: Specific design procedure for a geotextile filter

(Source: PIANC, 1987a)

This is represented diagrammatically in Fig.3.3-2. The vertical line for grain size equal to 0.06 mm is divided into three portions A, B and C. Portion A lies above 40 %, portion B lies below 15 % and portion C lies between 15 % and 40 %. The grading curve for soils falling into range A will either lie wholly to the left of this vertical line or pass through portion A of the line.

Curves falling into range B will either be wholly to the right of the line or pass through portion B only. Curves falling into range C will pass through portion C of the line.

In Fig.3.3-2 some typical grading curves are given as examples of these ranges.

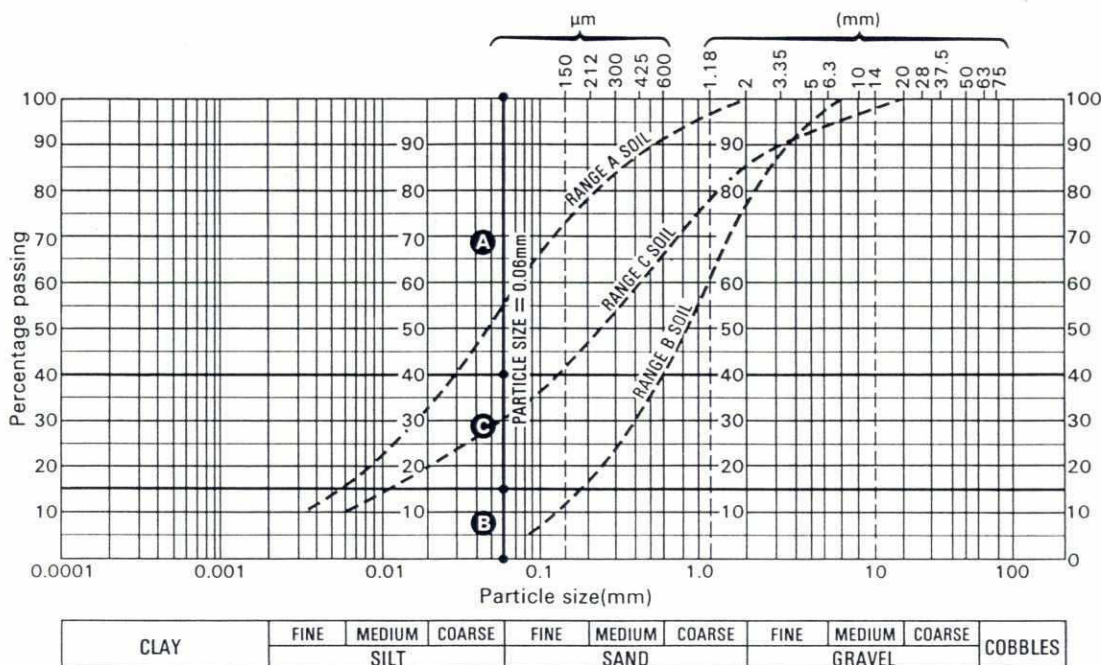


Fig.3.3-2: Soil grading ranges

(Source: PIANC, 1987a)

(2) Design for Soil Retention

Once the range has been established the filter can be designed for soil retention. The criteria for soil retention are based on the value of the effective opening size O_{90} . A knowledge of the test method used to determine O_{90} is vital when designing the filter.

The following rules given in Table 3.3-1 can only be used for O_{90} values determined by the wet sieving analysis of the Franzius Institute for Hydraulic Research and Coastal Engineering. This testing method is laid down in Swiss standards SN640550.

These rules only apply to dynamic load conditions (characterized by high turbulent flow and wave attack).

Grain size range	Retention criteria
A	$O_{90} < d_{90}^*$ $< 10 d_{50}$ $< 0.3 \text{ mm}$
B	$O_{90} < 1.5 d_{10} \sqrt{u}$ $< d_{50}$ $< 0.5 \text{ mm}$
C	As range B

* if the soil exhibits long-term stable cohesion then this rule may be relaxed to

$$O_{90} < 2 d_{90}$$

If the soil grading curve in range C is very flat then further investigation by laboratory testing is recommended; otherwise the criterion used for range B may be adopted.

Table 3.3-1: Soil retention criteria

(Source: PIANC, 1987a)

(3) Design for Permeability

Permittivity has been mentioned as a parameter useful in assessing the required geotextile permeability. However this approach calls for a knowledge of the hydraulic gradient in order to achieve an optimum design. At present this is often difficult to obtain and so the following is recommended as a practical alternative.

The geotextile filter must at all times maintain a permeability equal to or greater than that of the soil. Immediately after installation there will be a reduction in the permeability of the virgin fabric due to clogging and blocking. This reduction depends on pore structure and thickness of the fabric as well as the grain structure of the soil.

In general the permeability criterion requires that,

$$\eta k_g \geq k_s$$

where η = reduction factor
 k_g = permeability of the geotextile
 k_s = permeability of the soil

The value of the reduction factor depends to a large extent on the type of fabric.

1. for needlepunched non-wovens and other non-woven fabrics thicker than 2 mm (measured at a normal stress of 2 kN/m²),

$$\eta = \frac{1}{50}$$

2. For woven fabrics the reduction factor is dependent upon the permeability of the geotextile on the d_{10} of the soil, and can be found from Fig.3.3-3.
3. For thin non-woven geotextiles no specific criteria are available. However, the filtering behaviour of these fabrics is to some extent similar to woven fabrics and it is therefore suggested that Fig.3.3-3 is used if not other information is available.

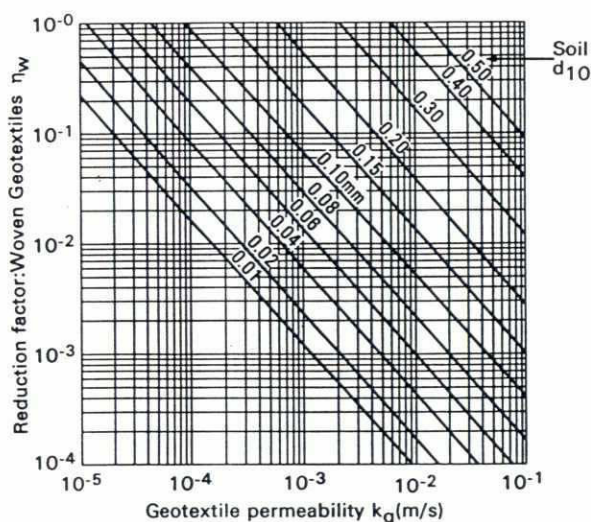


Fig.3.3-3: Permeability reduction factor

(Source: PIANC, 1987a)

(4) Identification of Soils Susceptible to Downslope Migration

The problem of downslope migration is most apparent in silts, sandy silts and fine sands. The minor cohesive properties of these types of soils tend to encourage high mobility of small single grains at relatively low hydraulic gradients.

The distinction between soils susceptible to downslope migration and those not susceptible is related to grain size, permeability and degree of cohesion.

The following procedure gives a general method of identifying soils susceptible to downslope migration. There may however be exceptions to the conditions given here and the designer must use his own judgement in such cases.

Soils susceptible to downslope migration will satisfy the conditions that a proportion of particles must be smaller than 0.06 mm. Additionally the soil will satisfy at least one of the following,

- (a) The coefficient of uniformity $U = d_{60}/d_{10} < 15$
The limit of 15 is a conservative estimate based on practical experience. For soils with a coefficient of uniformity greater than 15 a secondary filter will normally become established.
- (b) 50 % or more of the particles will lie in the range $0.02 \text{ mm} < d < 0.1 \text{ mm}$.
- (c) The plasticity index will be $I_p = w_L - w_p < 0.15$

The plasticity index is the difference between the moisture content at liquid limit (w_L) and the moisture content at plastic limit (w_p). If I_p is known at the preliminary design stage then the following criterion may be substituted,

$$\frac{\text{proportion of clay } (d < 0.002 \text{ mm})}{\text{proportion of silt } (0.002 < d < 0.06 \text{ mm})} < 0.5$$

If the soil is considered susceptible to downslope migration the design should proceed in accordance with para 5 otherwise the guidance given in para 6 should be used.

(5) Methods of Preventing Downslope Migration

There are three ways of preventing serious downslope migration.

(a) Attenuation of the hydraulic gradient

This can be achieved by incorporating a granular sublayer between the geotextile and the cover layer. In practice the granular material should be fine enough to provide an adequate damping effect yet coarse enough to be retained by the cover layer.

The sublayer should have a thickness of between 100 mm and 500 mm. For the conditions given at the beginning of this section 300 mm has been found to be adequate.

(b) Stabilization of the soil surface

Experience has shown that a thick layer of coarse fibers attached to the back of the geotextile filter integrates with the soil surface, thus reducing downslope migration of soil particles within the layer.

The physical requirements of this layer are dependent upon the range of the soil grading curve and are given in Table 3.2-2.

Characteristics of coarse layer	Range A grading curve	Range B grading curve
O_{90}	$0.3 < O_{90} < 1.5 \text{ mm}$	$0.5 < O_{90} < 2.0 \text{ mm}$
Thickness	$5 < t_{\text{ff}} < 15 \text{ mm}$	$5 < t_{\text{ff}} < 20 \text{ mm}$

Table 3.3-2: Requirements of the coarse layer

(Source: PIANC, 1987a)

Also practical experience has shown that the filter fabric should have a minimum thickness of about 5 mm.

(c) Application of a constraining load

This can be achieved by using a heavy weight cover layer which imparts a significant load into the bank slope thereby resisting uplift pressures caused by excess pore water pressures. As a practical means of preventing downslope migration either alternative (a) or (b) is suggested. Fig.3.3-4 illustrates alternative (a) and (b) together with the case where the soil is not susceptible to downslope migration.

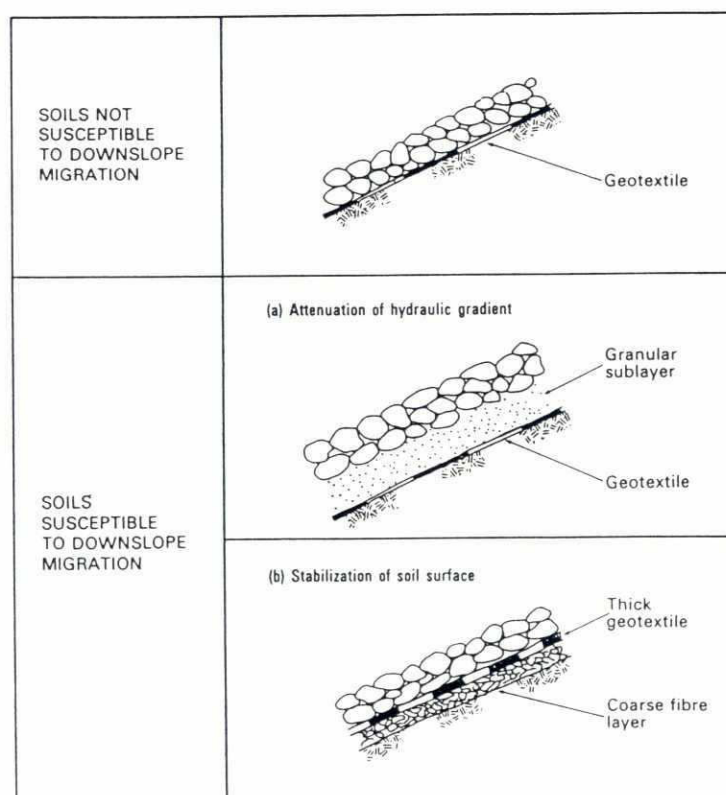


Fig.3.3-4: Recommended filter systems

(Source: PIANC, 1987a)

(6) Design aspects of Soils not Susceptible to Downslope Migration

For soils not subject to the problem of downslope migration the filter has only to meet the requirements related to soil retention (see para 2 above) and permeability (see para 3 above).

For situations where the engineer decides downslope migration could be a problem but not to the extent of requiring the measures described in para 5 he must then use his own judgement to make appropriate modifications to the recommendations in para 5.

3.4 DESIGN RULES OF THE FEDERAL INSTITUTE FOR WATERWAYS ENGINEERING (BAW), GERMANY

The most recent guidelines for the application of geotextiles in hydraulic engineering are published by the German Association for Water Resources and Land Improvement (DVWK, 1992). Other guidelines with directly applicable rules were issued by the Federal Institute for Waterways Engineering (Bundesanstalt für Wasserbau, BAW), Germany, under the title 'Applications of Geotextile Filters in Waterways' (BAW, 1989).

These recommendations are applicable to standard constructions of revetments incorporating geotextiles as shown in Fig.3.4-1.

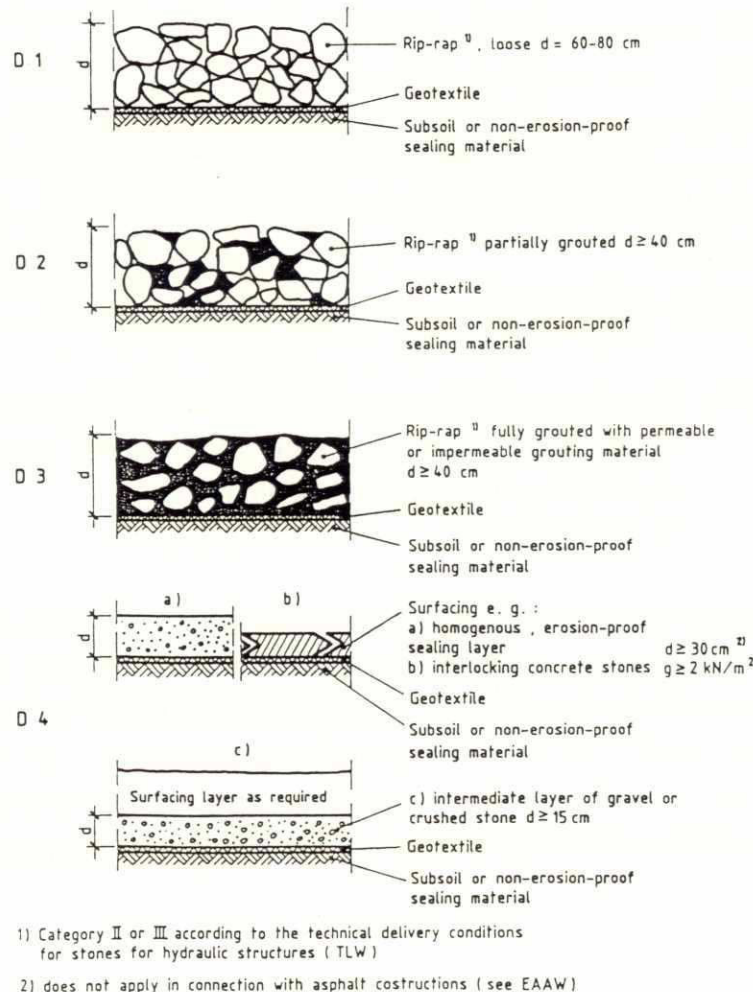


Fig.3.4-1: Standard constructions of revetments

(Source: BAW, 1989)

Based on experiences gained in the research centre of BAW, minimum and maximum values of requirements for geotextiles are compiled in Table 3.4-1 for different soil types to be retained (see Fig.3.4-2) and the standard constructions as per Fig.3.4-1.

In case of permeable revetments, the permeability criterion is fulfilled, in case the permeability coefficient of the geotextile is greater than shown in line 7 of Table 3.4-1.

Extract from the BAW leaflet

' Application of Geotextiles in waterways (MAG) '

D 1 = rip-rap , loose

D 2 = rip-rap + partial grouting

D 3 = rip-rap + full grouting ¹⁾

D 4 = surfacing

type of soil to be retained			type of soil 1					type of soil 2					type of soil 3					type of soil 4				
cover layer			D 1	D 2	D 3	D 4		D 1	D 2	D 3	D 4		D 1	D 2	D 3	D 4		D 1	D 2	D 3	D 4	
1	thickness of filter layer (s)	mm	4,5					4,5					4,5					6,0				
2	additional layer ²⁾	thickness						$D_w \leq 2 \text{ mm} : 5,0-15$ $D_w \geq 8 \text{ mm} : 15-25$					$D_w \leq 2 \text{ mm} : 5,0-15$ $D_w \geq 8 \text{ mm} : 15-25$					$D_w \leq 2 \text{ mm} : 5,0-15$ $D_w \geq 8 \text{ mm} : 15-25$				
3		effective opening size D_w						0,5 - 2,0 or 8,0 - 20					0,5 - 2,0 or 8,0 - 20					0,3 - 1,5 or 8,0 - 20				
4	minimum tensile strength according to DIN 53 857 longitudinal and transvers		N / 10 cm					1200/900					1200/900					1200/900				
5	soil particle retaining ability	permissible total volume of soil penetration	g					25					25					300				
6		as above , but in last testing phase	g					2,5					2,5					30				
7	hydraulic filter properties		m / s					$> 8 \cdot 10^{-4}$					$> 6 \cdot 10^{-4}$					$> 1,2 \cdot 10^{-4}$				
8	resistance to rupturing strength	rip-rap class II	Nm					> 600					> 600					> 600				
9		rip-rap class III	Nm					> 1200					> 1200					> 1200				
10	abrasion resistance		-	yes	—	yes ³⁾	yes	—	yes ³⁾	yes	—	yes ³⁾	yes	—	yes ³⁾	yes	—	yes ³⁾	yes	—	yes ³⁾	yes ³⁾
11	resistance to high temperatures up to 170° C		-	-	4)	-	-	4)	-	-	4)	-	-	4)	-	-	4)	-	-	4)	-	4)

1) Under impermeable surfacing layers (D3 , D4) the following applies for :

minimum thickness of filter layer : - -

additional layer : - -

hydraulic filter effectiveness : $k < 1 \cdot 10^{-4} \text{ m/s}$

mechanical filter effectiveness : values for soil type 4

other requirements : as in table

2) Application criteria , see point 4.2.4.1 MAG

3) Applies only to permeable interlocking stone layers

4) Only for hot bituminous coated materials

Table 3.4-1: Requirements for geotextiles according to the test recommendations of the BAW valid for standard revetment constructions with a slope of $m \geq 3$ and hydraulic loads

(Source: BAW, 1989)

In case the geotextile filter has to protect soil types which are not covered by those shown in Fig.3.4-2, the geotextile may also be used on condition that the grain size distribution of the soil to be retained is in the limits of the grain size distribution of soil types 1 to 4 as shown in the following Fig. 3.4-3 to 3.4-6, provided the coefficient of uniformity is less than 5 and the grain size distribution curve is steady. Moreover, in order to fulfil the permeability criteria the permeability coefficient of the geotextile filter k_g must be greater than that of the soil to be retained k_s :

$$k_g \geq 22 \quad k_s \quad \text{for non-cohesive soils}$$

$$k_g \geq 100 \quad k_s \quad \text{for cohesive soils}$$

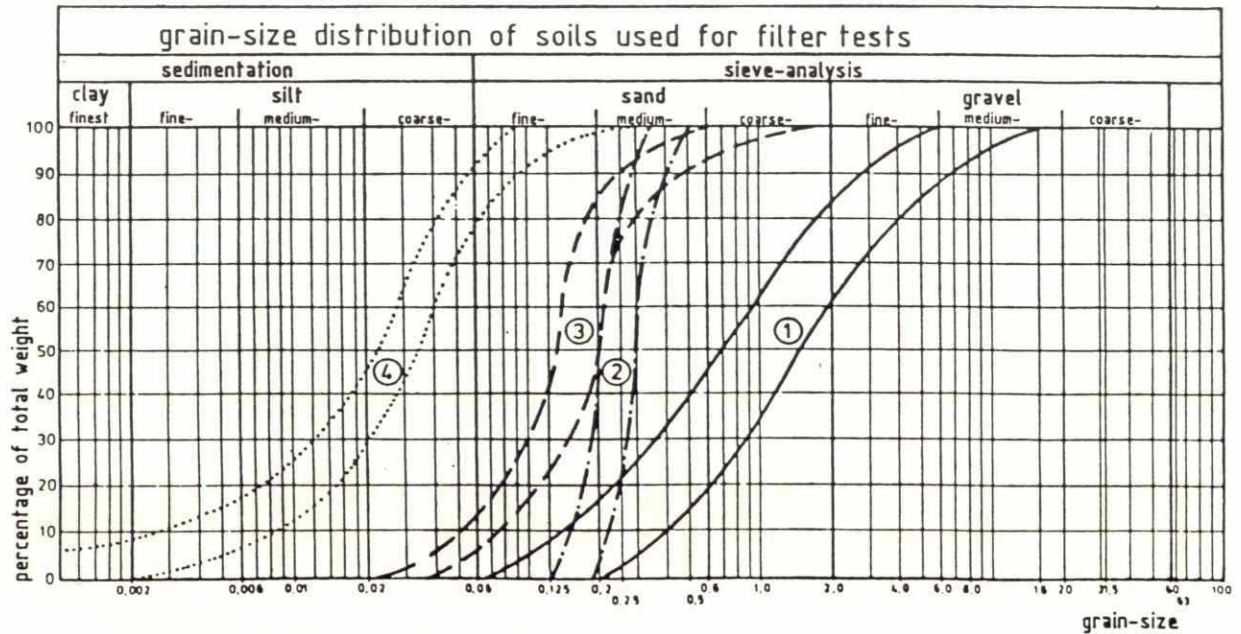


Fig.3.4-2: Grain size distribution of different soil types

(Source: BAW, 1989)

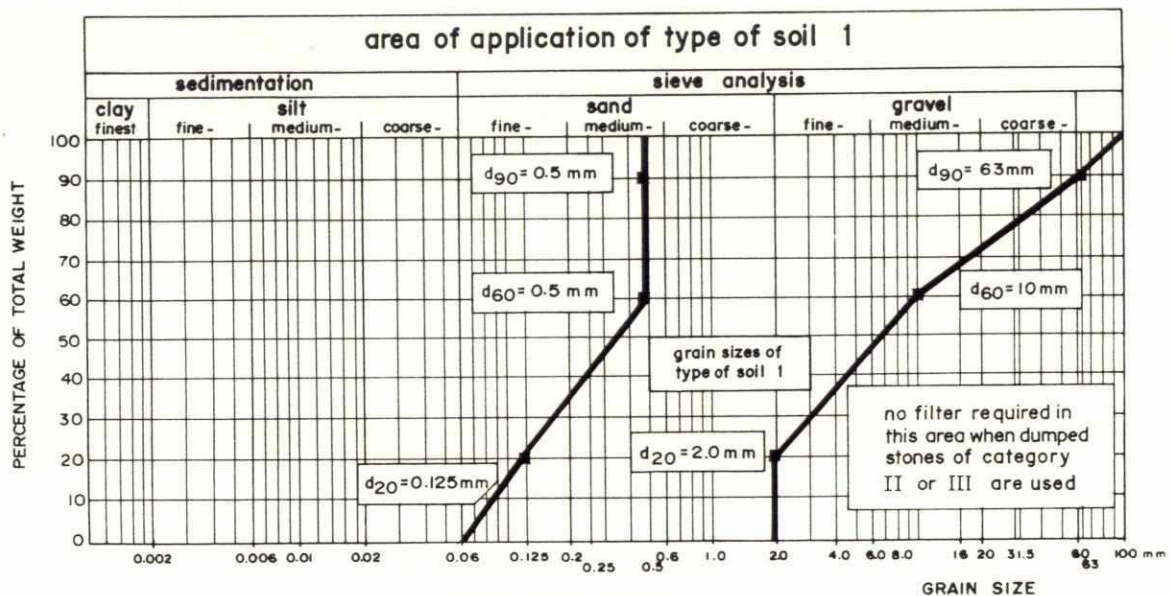


Fig.3.4-3: Type of soil 1

(Source: BAW, 1989)

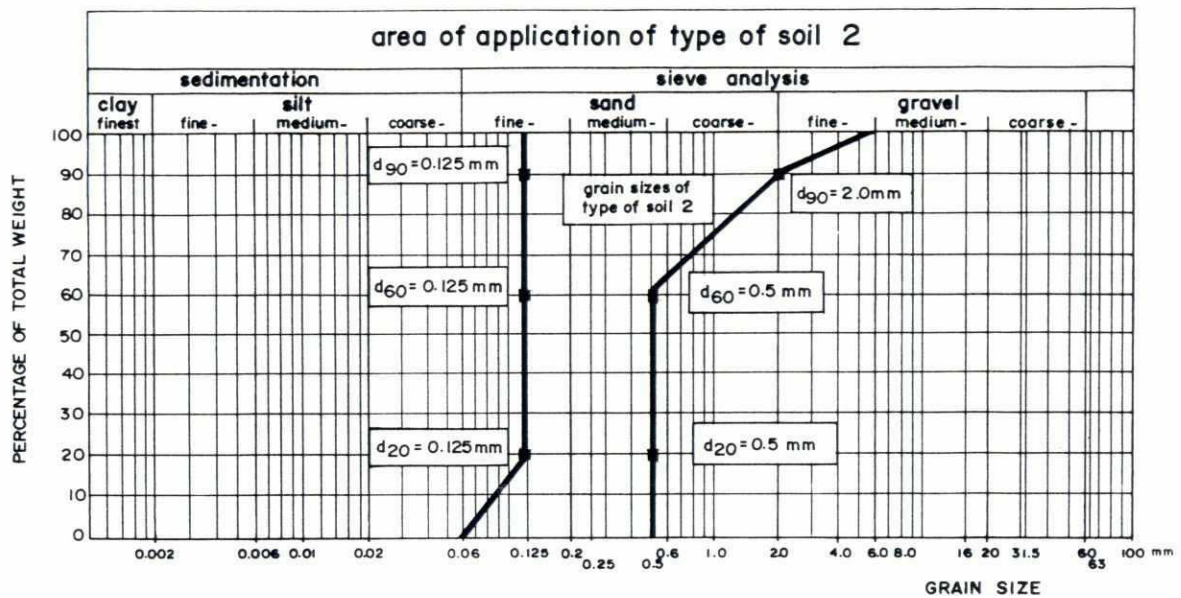


Fig.3.4-4: Type of soil 2

(Source: BAW, 1989)

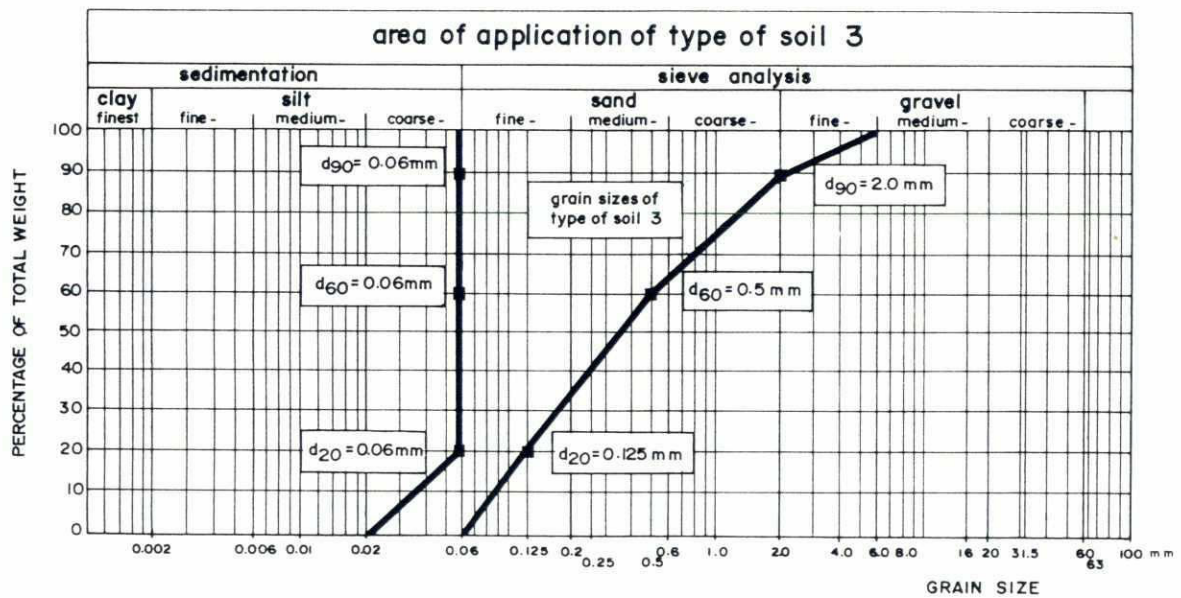


Fig.3.4-5: Type of soil 3

(Source: BAW, 1989)

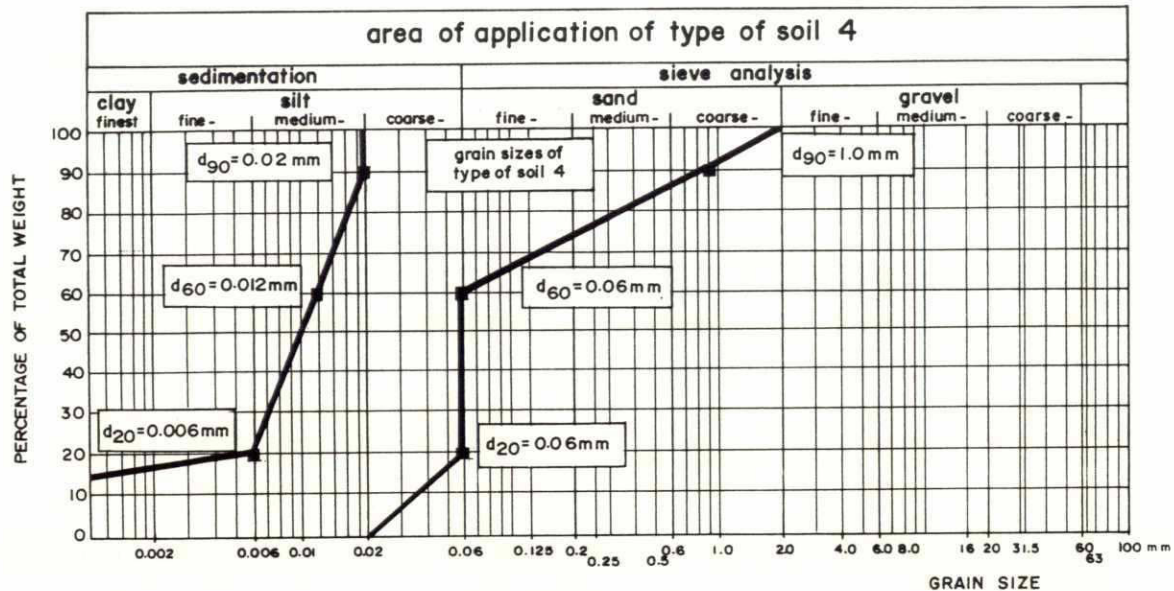


Fig.3.4-6: Type of soil 4

(Source: BAW, 1989)

3.5 DESIGN RULES OF THE GERMAN SOCIETY FOR SOIL MECHANICS (GSSMFE)

There are also recommendations of the German Society for Soil Mechanics and Foundation Engineering (GSSMFE), which deal with filter rules and problem soils, published in BAW, 1989.

The following criteria indicate independently problem soils with an increased mobility of fine soil particles (tendency to scouring, erosion and silting up):

1. Grain fractions $< 0.06 \text{ mm}$ contained in soil and: Coefficient of uniformity

$$U = d_{60}/d_{10} < 15$$

If $U > 15$, a secondary filter will probably establish itself. The limit $U = 15$ results from the practical use of drainage techniques. Under condition of well graded soils and minor hydraulic gradients a secondary filter is possibly built up even with lower coefficients of uniformity ($5 < U < 15$). Due to their tendency to internal erosion (washing-out of finest particles from the rough skeleton of the soil) soils with $U > 10$ can lead to permanent wash-outs, which possibly can only be controlled by a correspondingly strict design of the mechanical filter efficiency (imperviousness) without consideration of the effects of a secondary filter.

2. Quantity of soil in the range $0.02 \text{ mm} < d < 0.1 \text{ mm} > 50 \%$

With criterion 2 the fine soil particles of high mobility, however, without any considerable cohesive properties are included.

3. Plasticity index $I_p < 0.15$ (15 %)

Already small quantities of finest soil particles ($0.002 \text{ mm} > 5 \%$) characterize the properties of a soil. The properties caused by fine grained quantities can among others be described by the plasticity index I_p which is defined as follows :

$$I_p = w_L - w_p$$

If I_p is unknown, the clay-silt ratio may be taken into account in order to identify a problem soil:

$$\frac{\text{quantity of clay}}{\text{quantity of silt}} < 0.5$$

For certain types of soil, filter rules were developed by GSSMFE with regard to the retention criterion and the permeability criterion.

For the assessment of the retention function of a geotextile the following has to be taken into account:

- grain size distribution of the soil;
- effective loads (static/dynamic);
- type of structure, and
- safety requirements for the geotextile resulting from the structure.

Deviations from the following filter rules may be justified by assessment of the above specific conditions. Moreover, it is to be mentioned that the effective opening size $O_{90,w}$ of a geotextile has to be determined in accordance with the procedure specified in the recommendations of GSSMFE.

- (A) Filter rule for soils with $d_{40} \leq 0.06 \text{ mm}$ (soil type A), see Fig.3.5-1.

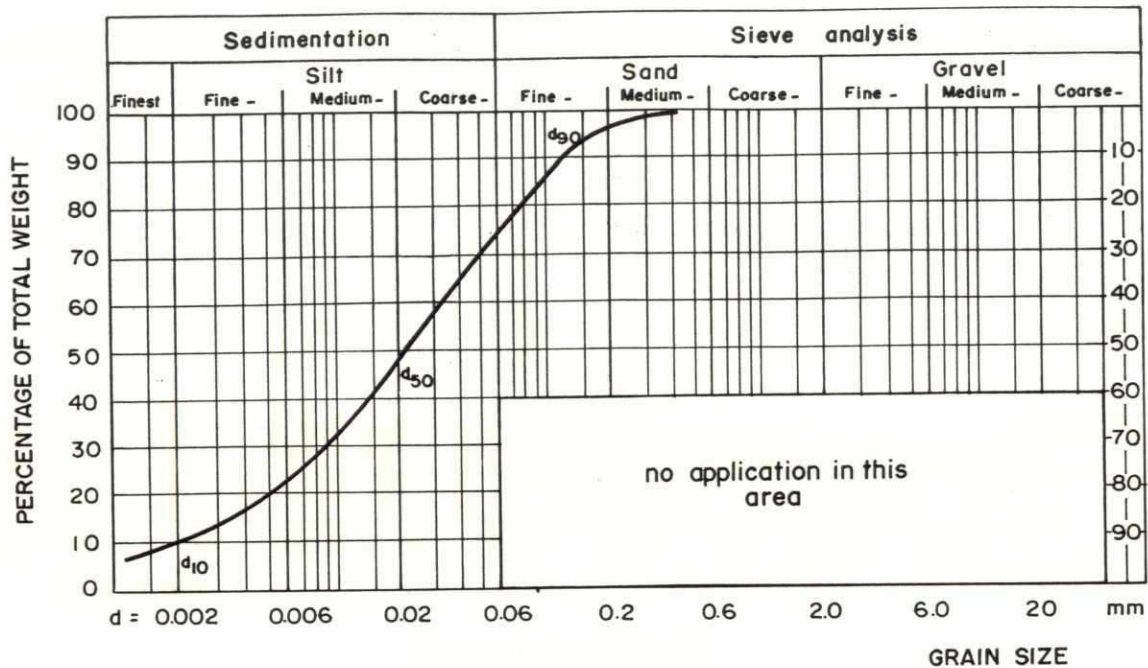


Fig.3.5-1: Soil type A

(Source: BAW, 1989)

- (a) static load condition: $O_{90,w} < 10 d_{50}$
- in addition for problem soils : $O_{90,w} < d_{90}$
 - for soils with stable cohesion in the long term: $O_{90,w} < 2 d_{90}$
- this rule may only be applied on condition that the soil to be retained will not lose its cohesive properties under loading.

- (b) dynamic load condition: $O_{90,w} < d_{90}$
- in addition it is recommended to keep $O_{90,w} < 0.3$ mm in order to guarantee sufficient retention capacity against the fine grains.

(B) Filter rule for soils with $d_{15} \geq 0.06$ mm (soil type B), see Fig.3.5-2.

- (a) static load condition: $O_{90,w} < 5 d_{10} \sqrt{U}$ and $O_{90,w} < 2 d_{90}$

in addition for problem soils: $O_{90,w} < d_{90}$

- (b) dynamic load condition: $O_{90,w} < 1.5 D_{10} \sqrt{U}$ and $O_{90,w} < d_{90}$

in addition it is recommended to keep $O_{90,w} < 0.5$ mm in order to guarantee sufficient retention capacity against the fine grains.

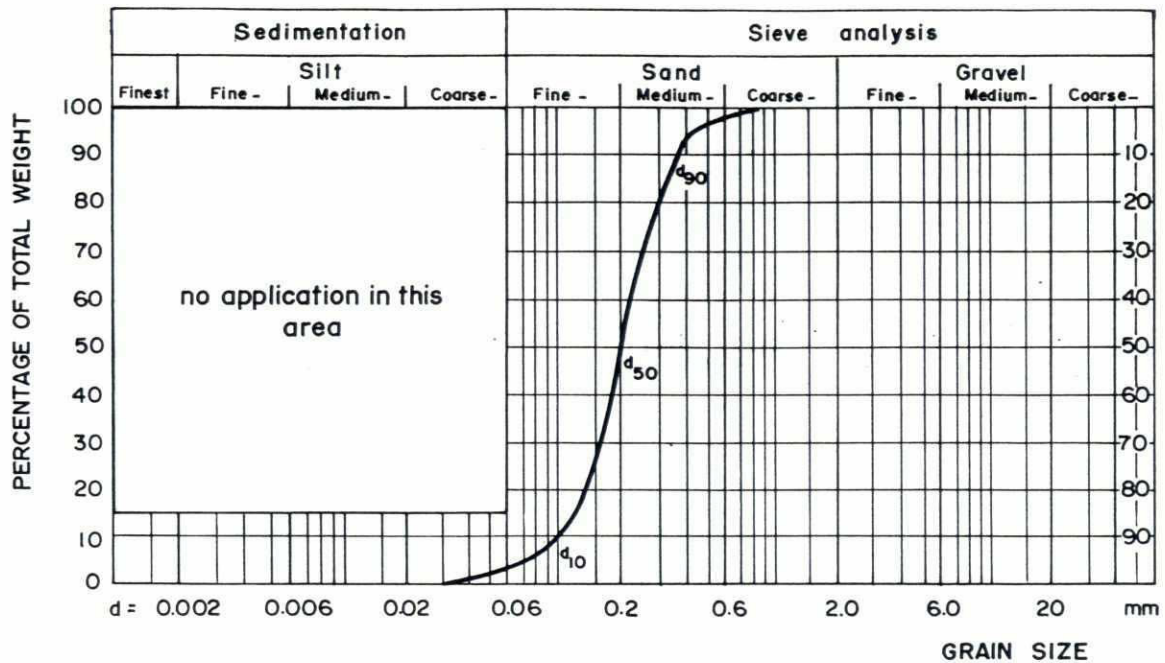


Fig.3.5-2: Soil type B

(Source: BAW, 1989)

- (C) Filter rule for soils with $d_{15} \leq 0.06$ mm and $d_{40} > 0.06$ mm (soil type C), see the following Fig.3.5-3:

For the soil type C the same filter rules are to be applied as for soil type B, however at the same time sufficient safety against suffusion is required.

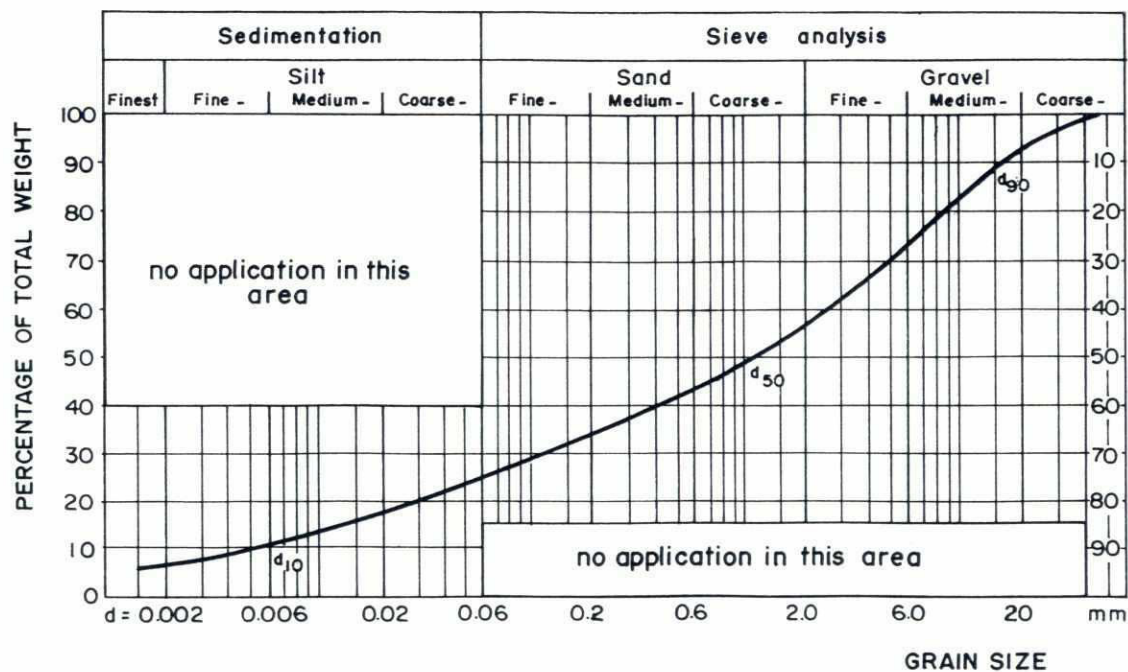


Fig.3.5-3: Soil type C

(Source: BAW, 1989)

With regard to the permeability criteria, it is required that the permeability coefficient k_v of the geotextile under a normal stress of 2 kN/m^2 multiplied with a reduction factor η must be greater than the coefficient of permeability k of the soil to be retained

$$\eta k_v \geq k \quad (3.5-1)$$

For the determination of the reduction factor η_G for woven fabrics and non-woven fabrics with a thickness of less than 2 mm, Fig.3.5-4 is applicable where

k_v = coefficient of permeability of the geotextile without soil contact
 d_{10} = grain size at 10 % of the distribution curve of the soil to be retained.

The results of investigations on non-woven fabrics bonded together in a mechanical process have shown that the permeability criterion for those geotextiles with a thickness of $30 * O_{90,w} \leq d \leq 50 * O_{90,w}$ is guaranteed in case silt or sand is to be retained and the permeability coefficient k_v of the geotextile measured on the fabric under a normal stress of 2 kN/m^2 is about 50 times greater than the coefficient of permeability k of the soil: $k_v \geq 50 k$

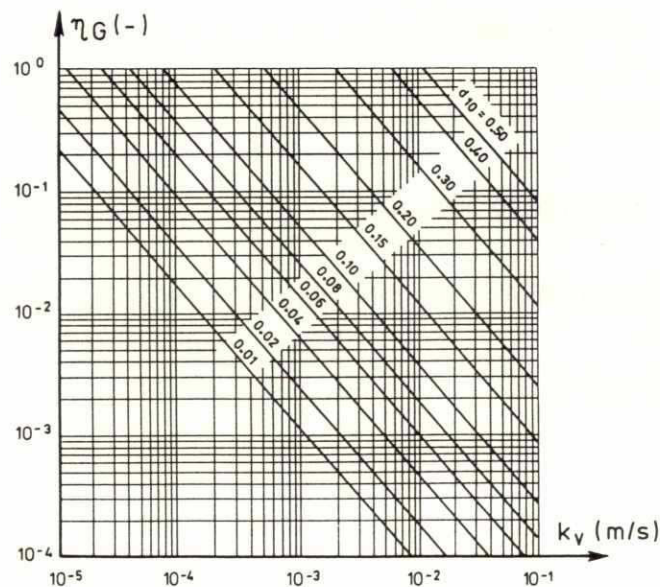


Fig.3.5-4: Permeability reduction factor η_G for woven fabrics and non-woven fabrics with a thickness less than 2 mm.

(Source: BAW, 1989)

The reduction factor η_v for sand and silty sand and mechanically bonded together non-woven fabrics with a thickness of more than 2 mm is given in Fig.3.5-5.

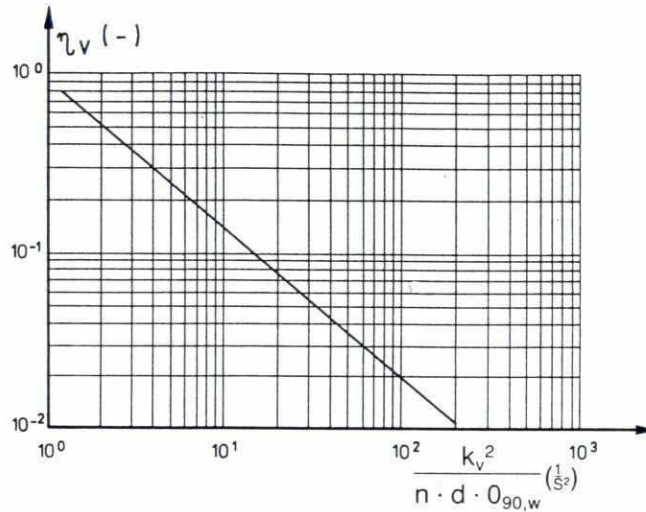


Fig.3.5-5: Permeability reduction factor η_v

(Source: BAW, 1989 and DVWK, 1992)

where

- k_v = coefficient of permeability of the geotextile without contact to the soil
- n = volume of pores
- d = thickness at 2 kN/m²
- $O_{90,w}$ = effective opening size.

The thickness of the selected geotextile should be

$$d \geq (25 \text{ to } 50) \cdot O_{90,w}$$

In case of dynamic load conditions the following thicknesses are recommended:

- for soil type A : $d \geq 6.0 \text{ mm}$
- for soil type B and C : $d \geq 4.5 \text{ mm}$.

REFERENCES

REFERENCES

AUTHOR/DATE	TITLE/PUBLISHER
BAW, (1990)	Guidelines for the Application of Geotextile Filters in Waterways, MAG (in German), Federal Institute for Waterways Engineering (BAW), Karlsruhe, Germany
DVWK, (1992)	Application of Geotextiles in Hydraulic Engineering (in German, English translation in 1993) German Association for Water Resources and Land Improvement, Verlag Paul Parey, Hamburg/Berlin, Germany
Hemphill, R. W. and Bramley, M.E., (1989)	Protection of River and Canal Banks, CIRIA, Butterworths, London, England
Heerten, G. and Wittmann, L., (1984)	Filtration Properties of Geotextiles and Mineral Filters, Demonstrated by an Example of Bank Protection, 23rd International Man-made Fibres Congress, Austria
Ingold, T. S., (1985)	A Theoretical and Laboratory Investigation of Alternating Flow Filtration Criteria for Woven Structures, Geotextiles and Geomembranes, Vol.2, No.1, 1985

APPENDIX



APPENDIX

SEMINAR ON FILTER TECHNOLOGY

September 5 and 6, 1992

INTRODUCTION

The pilot structures for bank protection include different types of revetments, consisting of a cover and a filter layer. Whereas the cover or armour layer is the primary protection against attacking forces of waves and current, the filter layer is equally important for the stability of the whole structure. There are numerous examples in the past where major expenditures for protection structures have been wasted as they failed completely resulting from insufficient filter layer. Specially such fine non-cohesive soils as found along the Jamuna river require proper protection against pumping and sucking wave and current action in the gaps of the cover layer, liquefaction, seepage and other sources of soil losses, leading to a collapse of the protection layer.

Whereas soil retention is the main object of applying a filter layer (mechanical filter stability), it must yet remain permeable to allow passage of water (hydraulic filter efficiency). However, these counteracting requirements are not the only determining factors in selecting the type of filter. Reliability for effective installation particularly below water and even in flowing water conditions, durability, uniform and controlled quality, strength in case of local failure of the cover layer are important considerations among many others. It is understood that the design must be economical, but cost is comparatively low in relation to the cover layer.

The objectives of the seminar were threefold:

- i) present the state-of-the-art in filter technology
- ii) discuss the available options with special reference to the use of jute and geojute in Bangladesh and
- iii) bring endusers, manufacturers and researches of geojute/jute products on one platform for interaction.

THE SEMINAR

The seminar was held as an internal event of the Consultant at Hotel Sheraton, Dhaka on 5th and 6th Sept., 1992. The detailed programme is annexed. Participants drawn from FPCO, BWDB and the Consultant, BJMC, BJRI, BUET presented an excellent mingling of endusers, manufacturers and researchers to discuss what is needed, what is being manufactured and what researches are going on for jute products as geonaturals. On the other hand, presentations by two internationally reputed and active specialists on geotextiles (both geosynthetics and geonaturals) and granular mineral filters made the filter technology understandable to the other participants. First day discussions were limited to the State-of-the-Art in Filter Technology. Prof. Dr R Floss and Dr G Heerten presented the followings papers on filter technology:

- i) Design and Dimensioning of Filters (Dr Heerten)
- ii) Revetment Design and Construction with Geotextiles (Prof. Floss)
- iii) Review of Geosynthetic Products and their Application (Dr Heerten)
- iv) Geotextiles - Risk of Damage During Installation, Field Investigations and Laboratory Results (Prof. Floss)
- v) Filtration Performance of Different Types of Geotextiles and Investigations on Geosynthetic Clay Liners (Prof. Floss)
- vi) Recent Development in Relation to Sand and Clay Mats as Composite Products of Textile and Mineral Component (Dr Heerten) and
- vii) The Use of Sand Bags and Sand Filled Tubes (Dr Heerten).

The presentations were thoroughly discussed by the participants.

The second day of the seminar was devoted to the discussions on geonaturals like geojute. Researchers on geojute from Jute Research and BUET took the floor to narrate their experiences and achievements in finding jute products that can be used in stabilizing soils on slopes. Different specimens from the laboratory of Jute Research were shown. The manufacturer Bangladesh Jute Mills Corporation (BJMC) also took floor to introduce their products. By way of discussion it was revealed that there was no coordination or interaction between the researchers and the manufacturers and the endusers are complete in the dark about the jute products that may have use in soil stabilizing action. The seminar, possibly for the first time, brought the endusers, manufacturers and researchers in geonaturals on the same platform to have dialogues along with the internationally reputed and active specialists in the field to fix up directions for future actions.

BWDB, Design Circle, NEZ, reported that during the last five years about Tk. 300 million worth of revetment works with boulders have been implemented by BWDB on the Meghna river and on other rivers in the NE region of Bangladesh. In these cases granular filters had been built which were quoted to have worked "admirably well above low water". The presence of problem soils - like in the Jamuna river - has been found a minimum. Filter

protection for falling apron were found not necessary. Due to missing equipment no post-construction surveys could be made up to now; they are scheduled to be undertaken by FAP 6 during the next dry season.

CONCLUSIONS

Although the seminar did not aim at achieving set targets but at discussing problems and possible solutions there are some conclusions that singled out of the event.

1. Design of Filters

It is paramount to keep in mind that the physics of filter technology are the same in granular and in geotextile filters. Hence the geotextile filters are to designed in analogy to granular filters. Several basic rules turned out to be important:

- filters (granular or geotextiles) and the cover layers have to be placed as to avoid any damage of the filter
- granular filters cannot be properly placed on underwater slopes, geotextile filters can
- the active components of filters are the voids not the grains (or fibers)
- the thickness of granular filters is governed by the procedure of placing the filter and their cover layer; the physically required thickness is far lower than the thickness required for proper placing
- filters (granular and geotextiles) have to be individually designed according to the specific boundary conditions, mainly the subsoil to be protected and the granulometry of the cover layer (in case of granular filters)
- elongation of geotextile filters is more important then tensile strength; the main strain on textile filters is due to placement
- geotextile filters below about 300 gr/m² shall not be used due to the risk of punctures and other damages, particularly below the water surface; further, lighter filters are generally not fulfilling the design criteria, especially not in case of silty fine sand soils.

2. Geotextile Materials

- The term geotextiles defines the difference to granular filters, it does not imply the type of materials. Hence geotextile filters may be made of synthetic or natural fibers.

- The decision on the type of fibers (geosynthetic or geonaturals) depends on the functionality and durability of the designed filter component.
- In all cases where long term durability is of minor importance, geonatural fibers can be used without problems.
- In many cases a composite filter made of geonaturals and geosynthetics promises to be the optimum solution in Bangladesh using the natural fibers to give the required strength for the placement procedures and the synthetic fibers for long term durability of the filter component.

3. Further Procedures

- It was decided that the endusers and the jute industry and research will cooperate with the aim to develop geotextiles fitting the requirements with regard to stability, strength, durability etc.
- Specifications and possible samples will be given to the Bangladesh jute industry/research for testing the manufacturing capacities and for a possible future production.
- The Project FAP 21/22, in its guidelines and manuals, will develop detailed criteria for the design of filters with special reference to typical Bangladesh boundary conditions. For the time being it is recommended to design geotextile filters as per the guidelines given in "Supplement to Bulletin No.57 (1987)" of PIANC which are at present the only international design rules.

FAP 21/22
Seminar on Filter Technology
Hotel Sheraton, Dhaka
September 05 and 06, 1992

SEPTEMBER 05, 1992

STATE-OF-THE ART IN FILTER DESIGN AND CONSTRUCTION

09.00 - 09.15	Introduction (Dr. Brühl)
09.15 - 10.15	Design and Dimensioning of Filters (Dr. Heerten)
10.15 - 10.25	Questions and answers
10.25 - 10.45	Tea/Coffee
10.45 - 11.45	Revetment Design and Construction with Geotextiles (Prof. Floss)
11.45 - 11.55	Questions and answers
11.55 - 12.25	Review of Geosynthetic Products and their Application (Dr. Heerten)
12.25 - 12.35	Questions and answers
12.35 - 14.00	Lunch
14.00 - 14.30	Geotextiles - Risk of Damage During Installation, Field Investigations and Laboratory Results (Prof. Floss)
14.30 - 14.40	Questions and answers
14.40 - 15.10	Filtration Performance of Different Types of Geotextiles and Investigations on Geosynthetic Clay Liners (Prof. Floss)
15.10 - 15.20	Questions and answers
15.20 - 15.40	Tea/Coffee
15.40 - 16.10	Recent Developments in Relation to Sand and Clay Mats as Composite Products of Textile and Mineral Components (Dr. Heerten)
16.10 - 16.20	Questions and answers
16.20 - 16.50	The use of Sand Bags and Sand Filled Tubes (Dr. Heerten)
16.50 - 17.00	Questions and answers
	Close

FAP 21/22
Seminar on Filter Technology
Hotel Sheraton, Dhaka
September 05 and 06, 1992

SEPTEMBER 06, 1992

DISCUSSIONS ON BANGLADESH PRACTICE IN FILTER TECHNOLOGY
AND NEEDS FOR REVETMENT DESIGN AND CONSTRUCTION

- | | |
|---------------|---|
| 09.00 - 09.15 | Introduction (Dr. Brühl) |
| 09.15 - 09.30 | Video Show |
| 09.30 - 12.30 | Panel Discussion on (with a tea/coffee break in between) |
| | - use of geotextiles in bank protection on rivers in Bangladesh |
| | - use of jute/geojute in bank protection |
| 12.30 - 14.00 | Lunch |
| 14.00 - 16.00 | Open discussion on the above subjects |
| 16.00 - 16.30 | Summary (Prof. Floss/Dr. Brühl) |

Tentative composition of Panel (up to 10 persons)

FPCO
BWDB
BUET
R&H
Jute Research Institute
Prof. Floss
Dr. Heerten
FAP 21/22

**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 18

DIMENSIONING OF GROYNES

JANUARY 1993

DIMENSIONING OF GROYNES

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1 INTRODUCTION

Different types of groynes are discussed in Subsection 4.2.4 just as their construction methods and materials in Section 5.2 of the Main Report. Accordingly groynes are stone, gravel, rock, earth or pile structures which are built at an angle to a river bank to deflect the water flow away from critical zones to prevent erosion of the bank. Different types are distinguished: groynes inclined upstream, set at right angles to the axis of the current or inclined downstream. 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.

The following design aspects are described: the type of groynes (Section 2), the length and spacing of the groynes (Section 3), the shape of the head of groynes (Section 4), the cross-section layout (Section 5) and the orientation (Section 6).

2 TYPE OF GROYNES

A first step in the design process of a bank protection with groynes 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.

Groynes can be used single or in series. Their use in series arises if a long reach is to be protected and depends also on their action 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 depth of scour. Permeable groynes have an advantage if placed in a series of groynes, because the effect of several permeable groynes on 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 lay-out of a series of groynes all the groynes are similar regarding length, cross-section, orientation and shape of the head of the groyne. However, often the most upstream groyne is attacked by the flow stronger than the other downstream groynes, and in that case 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 topplayers can be made stronger than those of the other downstream groynes.

Single groynes protect the bank downstream of the groyne over a relatively short distance against bank erosion. In Bangladesh they are generally applied to protect valuable infrastructure, as a town, an intake, a regulator or a pumping station and need to be impermeable and rather long to be effective. Due to their sensitivity to changing 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 groynes should not be submerged. However, if the maximum effectivity of the groynes is required during lower discharges, a submerged groyne can be cost effective, for example to improve the navigation depth during low discharges. Often a strong toplayer of the crest of a submerged groyne is needed to resist the strong attack if the crest is just submerged. This is especially true for relatively long groynes (blockage 20 % or more of the width of the cross-section).

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

3 LENGTH AND SPACING OF GROYNES

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

The width of the cross-section of the channel at bankfull stage should be blocked partially by a groyne to deflect the flow from the existing bank, if no more bank erosion is allowed or acceptable. Generally the length of the groyne is selected between 10% to maximum 50% of that width. In the wide and shallow channels of the Jamuna river the length is often selected around 5 to 15% of the width of the deeper part of a channel (not including the attached char land). The length is often limited by the wish not to construct the groyne in the deepest part of the channel close to the thalweg, because the costs per linear meter for construction in deep water are high. Along outer bends often series of relatively short groynes are built, because of the great water depths along the outer bend (bend scour and local scour).

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

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

Experiences with groynes in Indian rivers show that a blockage of 30 % of the channel width at bank full stage can be used as a guideline (Varshney et al., 1977 and Joglekar, 1971), see Fig.3-1.

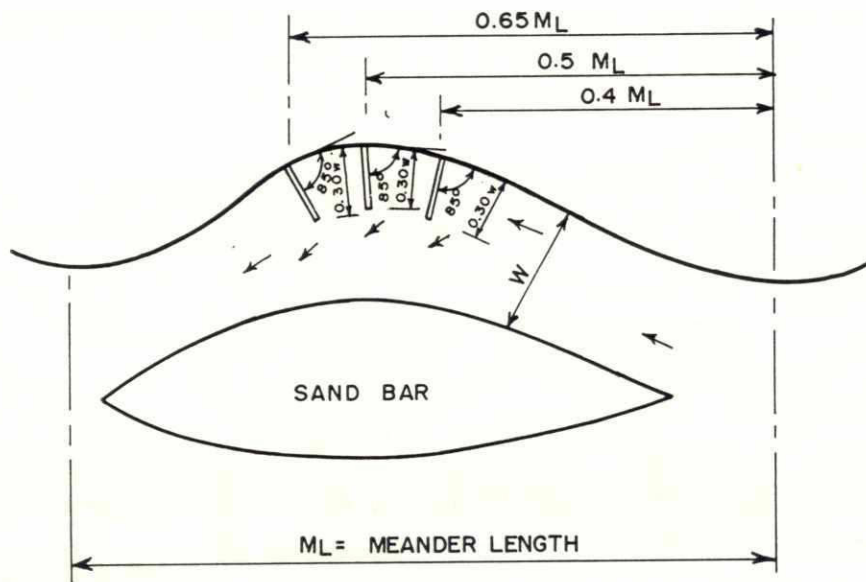


Fig. 3-1: A standard design based on experiences with groynes in Indian rivers

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

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

General practice is a spacing of about 2 to 3 times the length of groynes. This rule includes some safety for the bank erosion. In favourable conditions and without additional safety a single groyne can protect 4 to 5 times the length of the groyne, (see report of Meghna River Short Term Study, model investigation of the single groyne near Eklashpur). An Indian guideline suggests a spacing of the groynes by 0.1 to 0.15 of the meander length of an

outflanking channel. If the meander length is 15 to 30 times the channel width and the length of the groyne is 30 % of the channel width, then the spacing is 0.6 to 1.5 times the length of the groyne. For the Jamuna river conditions this guideline is on the safe side.

As an example the spacing of a series of impermeable groynes an outerbend of the upper Teesta river in India is shown in Fig. 3-2. There the spacing is designed as 1.1 to 2.2 the length of the groyne measured up to the embankment. To be safe, the embankment is retrieved from the actual bankline. The ratio between the effective length of the groyne up to the actual bank line and the groyne spacing varies between 2 and 3.7. Since the Teesta river is also a braiding river, these results can be applied also for the Jamuna river.

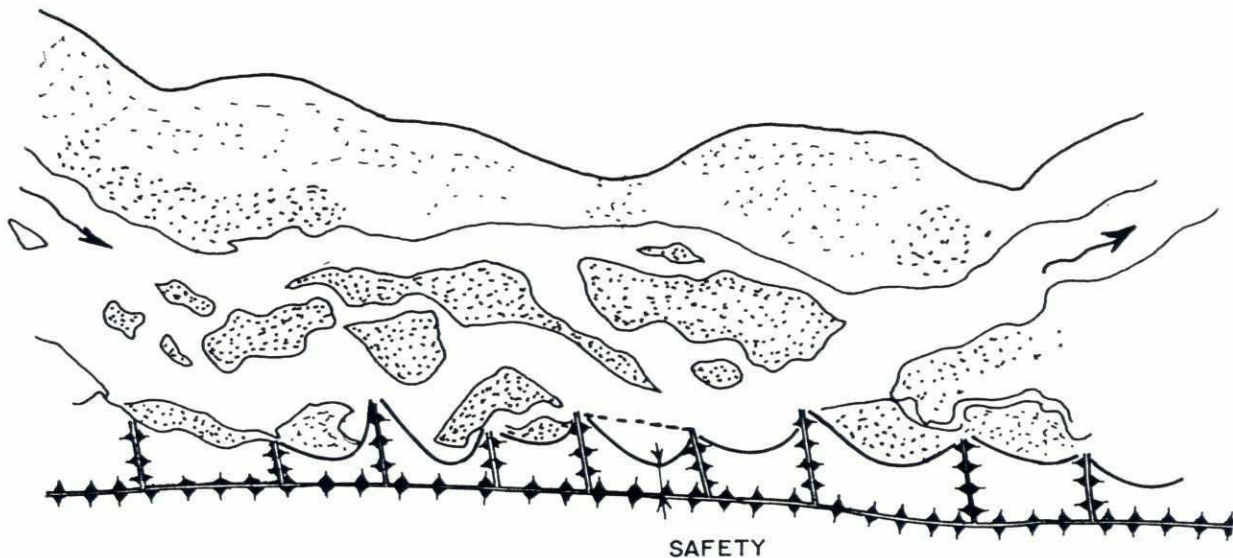


Fig. 3-2: Layout of groynes along the upper Teesta river

Another example is the preliminary design of hard points, which are comparable with T-head groynes, at Sariakandi/Mathurapara and Kazipur by FAP 1. The design ratio of spacing versus groyne length ratio varies from 2.5 to 3.6 and the ratio based on the expected actual bankline varies from 3.5 to 4.3 along the straight bank in Sariakandi and 3.0 in the sharp bend near Kazipur. From a comparison of these values it is seen that an expected safety of 0.7 to 1.0 is included in the design ratio.

The combined permeable/impermeable groynes tested in the physical model investigations of the present study showed good results for a spacing of 2.5 times the length of the groynes.

Hence, 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.

Often groynes are not only intended to protect the banks from being eroded but also, particularly in alluvial rivers which are less wide than the major channels of the Jamuna, to increase the depth of the navigation channel by reducing the low discharge channel width. In these cases, an optimal increase of the navigable depth is obtained if the spacing is 2 times the length of the groynes, because with that ratio just one eddy between two groynes gives a maximum guidance to the main flow in the river. If this ratio is increased then that eddy breaks into 2 or more smaller eddies which create a more instable main flow for the navigation.

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.

4 SHAPES OF THE GROYPE HEADS

Different riverside ends of the groynes, called groyne heads, are being applied, mainly to reduce the scour of the riverbed. Fig.4-1 shows the different types of scour, notably local scour, protrusion scour and constriction scour at a straight groyne head which is the most unfavourable design in that respect (but also the less expensive one). Constriction scour is observed in cases that straight groynes reduce the channel width by more than 10 %. The constriction scour generally develops in the front of the head, see Fig. 4-1. At the downstream side of the head a local scour hole develops. If the flow at the upstream side of the groyne accelerates gradually a protrusion scour hole can develop at the upstream side of the head of the groyne.

The main types of groyne heads of impermeable and not-submerged groynes are described in the following; see also Fig. 4-2.

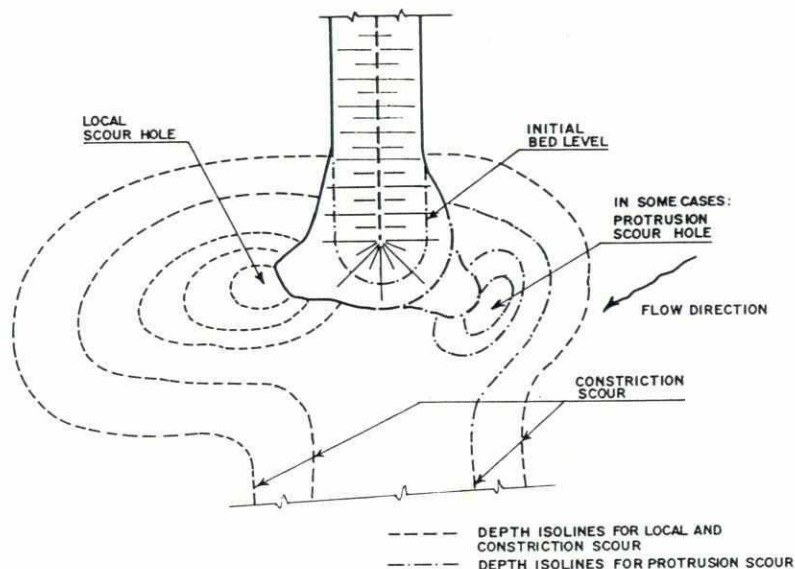


Fig. 4-1: Scour around the head of a groyne

Bell-head

A strongly protected head is also called a bell-head. The separation point of the flow can shift easily along this head and consequently the direction of the vortex street downstream of this separation 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 define 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 more than 30 %) a T-head reduces the risk of erosion at the opposite bank downstream of the groyne. A long upstream wing can reduce the attack on the shank of the groyne and allow a reduction of the strength of the toplayer of the shank. The wings have in general a strong toplayer. In case there is only one wing they are called L-head groynes. The effect of the latter ones can be to have larger sediment deposits between the groynes, to provide a better protection to the bank and to have less scouring at their head. T-head or L-head groynes are often built when the direction of the approach flow upstream of the groynes can change due to the morphologic development of the river channels.

The concept of hard points, as developed and proposed by FAP 1 is comparable to short T-head groynes.

Hockey-stick head

Groynes bent like a hockey stick at their head are known as hockey shaped groynes, which is a combination of a short L-head with a bell head. Along the upstream side of the hockey

stick the flow separation point can shift but the alignment to the flow reduces the intensity of the vortex street and also reduces the local scour depth. This shifting of the flow separation point is generally not attractive because the flow pattern downstream of the groyne is not well defined. A hockey-stick is applied if a very strong attack on the head is expected, stronger than in case of a bell-head, and if a small reduction of this attack allows the selection of a less strong toplayer of the head.

Design rules

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

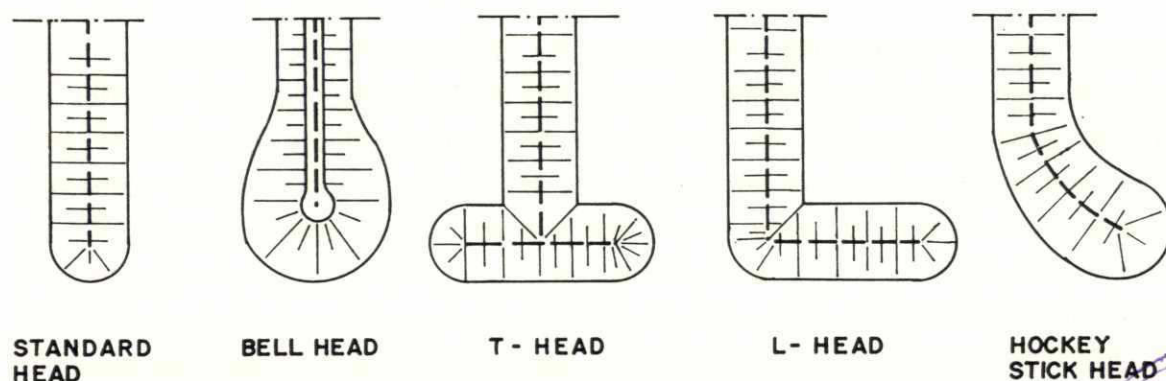


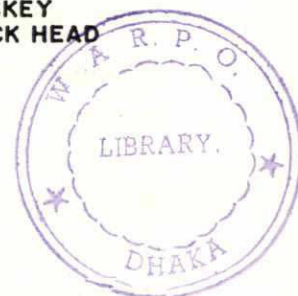
Fig. 4-2: Shape of groyne heads

5 CROSS-SECTION

A typical cross-section of a groyne depends on many considerations, one of the most important ones being whether the groyne should be conceived as permeable or impermeable. The cross-section of a permeable groyne is described in Section 5.1 and the cross-section of an impermeable groyne in Section 5.2.

5.1 PERMEABLE GROYNES

Permeable groynes are generally constructed of wood, 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 horizontal 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, see Fig.5-1. No recommendation is given for a certain arrangement.

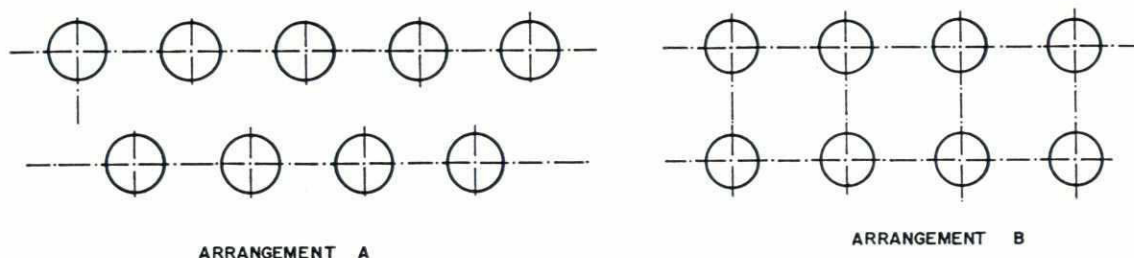


Fig.5-1: Two layouts for positioning parallel pile rows

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 m 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 rows 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.

5.2 IMPERMEABLE GROYNES

Vertical slopes can for example be made of steel or concrete sheet piling as a part of a cofferdam or of gabions. The space between the vertical walls may be filled with locally available material such as sand, gravel etc. applying a (geotextile) filter where required. The design of the individual components of the structure (sheet piles, anchoring, gabion wall) has to follow the standard rules of structural and foundation engineering. The vertical walls can induce serious scouring of an alluvial river bed and require a massive bed protection around the head, since downstream of a groyne with a vertical head strong eddies with high flow velocities are generated. For this reason vertical groynes are not recommended as a standard design.

Therefore, standard cross-sections of the shank of an impermeable groyne have a trapezoidal shape: two side 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 toplayer their slope should be designed as steep as the stability of the toplayer allows to minimize the area of the toplayer. 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 the 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 this 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 toplayers of inclined slopes are often different above and below SLW. The toplayer 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 SLW may be steeper to save space and construction material. The transition from one toplayer design to another is often less resistant to flow attack than the actual toplayers. Therefore a berm with a width of at least 1.5 m is designed at that 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 toplayer elements. For example the spaces between concrete blocks of a toplayer should be limited to a certain maximum value to safeguard 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 gradually 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 in case of an impermeable groyne. In a special case, e.g for deep water and/or strong flow and wave attack, a steeper slope of the crest may also be appropriate.

It must be stressed that the above mentioned slopes are not only depending on the hydraulic loads, but also on the construction material of the core and the protection of the slopes. In any case detailed stability investigations are required in order to establish the required slope, as well as estimations of settlements of the earth built dam depending mainly on the subsoil conditions.

The existing river bed around a groyne should be protected from scouring by a bed protection or a falling apron. A permanent bed protection has a length of about 15 times the

scour depth at the downstream edge of the bed protection. In practise this length proved to be sufficient to guarantee the stability of the groyne. For the design of the falling apron there are two different approaches notably either a rather short but thick one or a long and thinner one. The short and thick falling apron follows the development of a scour hole by falling and sliding into it; therefore all type of materials can be used. The long and thin falling apron is designed to allow the subsoil sand to be sucked out from underneath the blocks or boulders of the toplayer. This requires a certain permeability which can be achieved by a toplayer of uniform thickness and a uniform size of the elements (blocks or boulders). In Fig. 5-2 a sketch of the different types of falling aprons is presented. Since the physical processes occurring at falling aprons are not yet fully understood and since there are no proven design rules for application in fine non cohesive bed material, the Project intends to undertake special model tests for the detailed design, during the Test and Implementation Phase, to investigate the phenomena at different scales.

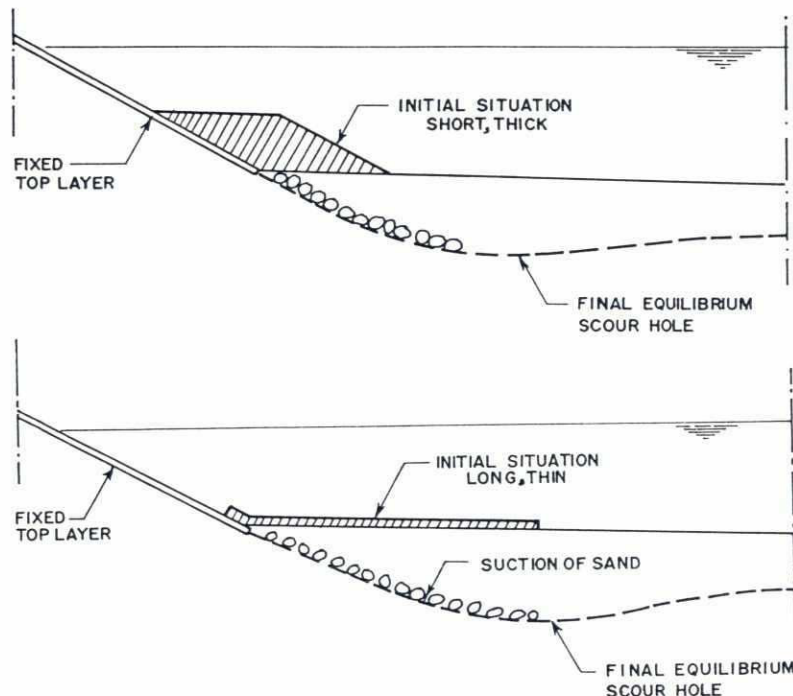


Fig. 5-2: Different types of falling aprons

6 ORIENTATION OF GROYNES

Groynes facing downstream are not suitable for bank protection surfaces, since the current may attack the root of the next downstream groyne endangering not only the root and the surrounding bank area, but the whole groyne itself. Groynes placed normal to the flow may protect only a small area. However, groynes facing upstream deflect the flow away from the bank, and they are able to protect bank areas upstream and downstream of themselves. Thus deflecting groynes seem to be best suited for bank protection or sedimentation purposes. Design guidelines recommend angles of 100° to 120° to the flow, however, also the form of the bank, that means concave or convex, must be taken into consideration. In straight reaches groynes may be pointed upstream as much as 105 to 110° .

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**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 19

CONSTRUCTIONAL ASPECTS

JANUARY 1993

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1 INTRODUCTION

Experience has shown that the traditional construction methods in Bangladesh are only partially suitable for the protection of the river banks consisting mostly of fine cohesionless soils against the high flow velocities. It is one of the objectives of this study to improve the traditional methods and to propose new methods supplying the latest state-of-the-art. Combinations of traditional and new methods are to be investigated in order to achieve most economical solutions taking into account the prevailing local conditions and checking the applicability of new methods under local conditions. The suitability of local construction material and their availability in sufficient quantities for the planned bank protection works are to be looked into. Furthermore, new construction material that is not available in Bangladesh is to be checked and verified for use under the prevailing conditions.

New construction material and methods as well as their use in combination with traditional construction methods may require particular equipment, the availability of which is to be investigated. Also the capability of local contractors to work with new materials and to apply modern construction methods in bank protection works had to be assessed.

2 CONSTRUCTION MATERIAL

2.1 GENERAL

Alternative solutions for bank protection systems in the form of revetments and groynes are described and discussed in Subsection 9.1.2 and 9.1.3 of the Main Report, respectively. For the realization of those proposals a great variety of different construction materials is required.

The construction of groynes at Kamarjani requires mainly:

- steel piles, concrete piles, steel sheet piles but also smaller amounts of some of the materials mentioned below for Bahadurabad.

For the construction of the various types of revetment at Bahadurabad more or less large amounts of the following material will be needed:

- sand, gravel, stones, boulders, cement, concrete, bricks, bitumen, geotextiles, wire mesh etc.

The assessment of the materials has, further to the question of availability, also to include the investigation of their quality with regard to suitability for high standard construction works.

2.2 CONCRETE AGGREGATES

2.2.1 General

Concrete may be required for the production of individual blocks, cable connected block mats and mats consisting of geotextiles to which concrete blocks are fixed.

Any concrete has to be designed to have the strength, durability and density required for its specific utilization. These concrete properties depend on the amounts and quantities of the materials required to produce the concrete, such as cement, aggregates, mixing water.

2.2.2 Cement

Cement is produced in Bangladesh in the Sylhet and the Chittagong area, however, the two existing cement plants are not able to cover the total market demand. Moreover, one brand of cement is of poor quality only, whereas the produced quantity of the higher standard cement is reported to be rather limited. Since the total quantity of local made cement seems to be used by Government authorities, it is unavoidable to import from other countries where good quality cements are available: Malaysia, Indonesia, India, China and others. In general, however, the procurement of cement is not a problem in the country.

2.2.3 Mixing Water

According to chemical analyses made in the Jamuna Bridge Study, the Jamuna water can be used at the construction sites as mixing water for concrete production. This is, however, to be examined on the specific sites. In case the quality of surface water would not comply with relevant standards, ground water has to be used.

2.2.4 Sand

In the course of investigations and studies for the Jamuna Bridge Project the suitability of Jamuna river sand for fine aggregates has been investigated. Requirements which are to be fulfilled are the grading, composition and harmful substances. It was found out that Jamuna river sand can only be used as an admixture to other sands that are lacking the finest fraction e.g. the fraction smaller than 0.25 or 0.30 mm.

Furthermore, specifications and regulations limit the amount of very fine material smaller than 0.075 or 0.063 mm, since the water demand of concrete mixes is increasing considerably with the content of finest particles such as clay and silt. It was found that the river sand contains too much clay and silt, but also too high amounts of mica in the form of muscovite. In Annex 1 to the report on the Jamuna Bridge Bank Protection Study Project (1988) it is reported that the mica content varies from 6 to more than 28 %. Since the use

of micaceous sands will result in a loss of concrete strength it has been strongly advised not to use Jamuna river sand for concrete blocks or any other concrete works in the Jamuna Bridge Project, not even for small mortar blocks used in gabions or for pumped cement/sand mortars to fill fabric mattresses.

A possible option for the production of concrete and mortar is the use of sand and coarse aggregates available from other areas of Bangladesh as in Domar area in Dinajpur District and Kaliakoir area in Dhaka District. According to Hansen (1988) the use of sand from Fazilpur in the Sylhet district is acceptable for quality concrete. However, it should be taken into consideration that even the small amount of 2.6 % of mica contained in Fazilpur sand will lead to a reduction of the compressive strength in the range of 20 % which can partly be compensated by using more cement. For the Jamuna Bridge Project Fazilpur sand has been accepted only in case the mica content is limited to about 3 % and only if the mica consists of biotite and not of muscovite.

2.2.5 Coarse Aggregates

Sufficient quantities of gravel and boulders are available in Bangladesh, although not at the test sites of Kamarjani and Bahadurabad, but from the Sylhet district. It is common practice in Bangladesh that coarse aggregates needed for concrete production are obtained by crushing boulders by hand or crusher. That crushing is generally been done close to the site.

2.3 STONES AND BOULDERS

Boulders of sufficient size and weight are required for the construction of rip-rap and to fill wire mesh gabions and mattresses, but also for toe protection in the form of falling aprons. In the low water season this material is collected by hand from river beds at their entry to Northeast Bangladesh. Thus their availability depends on the "production" of the rivers, but direct imports from India and Bhutan are also common practise.

Hansen (1987) reports on reserves of natural stones and boulders of about 6.7 million m³ in Sylhet area and about 3.4 million m³ in the Panchagarh-Tetulia-Patgram area. Out of the above mentioned total quantities, about 680,000 m³ and 85,000 m³ per year are extracted from the two sites, respectively. However, the production could be increased considerably by use of modern and suitable equipment. In this connection mention should be made that according to the information given by dealers of construction material, a shortage of boulders or bottlenecks in their delivery combined with a price escalation are most likely to be expected in the near future as soon as the construction of the Jamuna Bridge and/or the River Bank Protection I Project (on the Jamuna and the Meghna) would start.

2.4 NATURAL ROCK

Natural crushed rock of sufficient size and weight required for slope protection is available in Bangladesh only in the Chittagong Hill Tracts. According to the investigations for the Jamuna Bridge Project the durability and density are expected to be sufficient. However, the production capacity of the quarries and hence the quantities which can be delivered within a short time are limited. The same holds for rock to be imported from India or Malaysia.

2.5 BRICKS

Bricks can be used to fill wire mesh mattresses and gabions. They should be of 1st class quality and the standard size is 25.4 * 12.7 * 7.6 cm. They should have sharp edges and be free from cracks, chips and lumps. The required minimum compressive strength is in the order of 10.5 N/mm². No problems in terms of quantity and quality of bricks are expected for the Project although the environmental hazards encountered in their production make bricks increasingly less attractive.

2.6 CHEMICALLY BONDED SAND BLOCKS

Manufacturing bricks without burning has been developed during the last 2 years in China. The Consultant has investigated this material by a special mission of his Construction Management Expert to China. According to his report the components of the blocks are:

- 67-70 % sand
- 20 % clay
- 5-8 % industrial residuals (fly ash, acetylene or fertilizer production residuals)
- 4 % cement
- 1 % chemical product (not specified)

After mixing the ingredients mechanically the blocks are formed by vibration under high pressure in special machines. Strength and density of the blocks are reported to be adjustable so that compressive strengths in the range of 7.5 to 20 N/mm² can be achieved, which could be further increased by adding rock material or boulders. The dry density is reported to be 1.8 t/m³. A special advantage would be that the mica content of the sand is reported not to adversely affect the strength of the blocks, so that local sand could be utilized without limitations.

The largest dimensions of blocks made until now are 40 * 40 * 25 cm. It is planned to take up the production of these chemically bonded sand blocks in Bangladesh in March 1993. Though savings seem to be possible in comparison with concrete blocks, detailed tests of strength and, in particular, longterm durability are required before using this newly developed material for the test structures. Further their environmental harmlessness should be proven by the manufacturers.

2.7 BITUMEN AND BITUMINOUS MATERIALS

2.7.1 General

Bitumen is produced in Bangladesh and available in sufficient quantity. Out of the possible revetments listed in Subsection 9.1.2 of the Main Report the following bituminous systems have been selected:

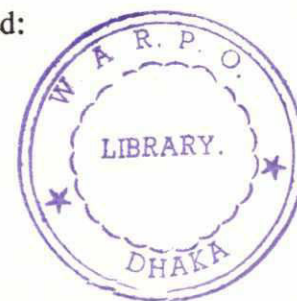
- grouted stones;
- open stone asphalt, and
- lean sand asphalt.

Various bituminous mixtures for use in hydraulic engineering are available, and for each application a special mix has to be designed. In general, the mixtures consists of bitumen and a mineral aggregate which is composed of sand, gravel, crushed stone and/or filler. The most important factor which influences the properties of the mixture is the bitumen content. The bitumen coats and binds the mineral particles in the aggregate mix. If more bitumen than necessary for coating and binding is added, the voids will gradually be filled.

Mixtures in which the bitumen serves only as a binder are called "underfilled mixtures". Their properties are directly related to the properties of the stone skeleton. With increasing bitumen proportion the voids in the mineral mix become filled and the influence of the bitumen on the mix properties is increased, whereas that of the stone skeleton is reduced. In mixes in which the voids are almost filled with bitumen, both the stone skeleton and the bitumen contribute to the properties of the mix. These mixtures are referred to as "nearly filled mixtures". They must be compacted, either mechanically or under their own weight. In case the volume of bitumen is larger than that of the voids in the mineral aggregate, the mixtures are called "overfilled mixtures". In that case the properties of the bitumen become predominant.

In hydraulic engineering the following main types of mixtures are distinguished:

- | | |
|---------------------------|-----------------------|
| - underfilled mixtures: | . lean sand asphalt |
| | . open stone asphalt |
| - nearly filled mixtures: | . asphaltic concrete |
| - overfilled mixtures: | . dense stone asphalt |
| | . asphalt mastic |
| | . grouting mortar. |



Another important distinction is to be made between mixtures which are either

- water impermeable or
- water permeable,

depending of the bitumen content. The higher bitumen content is, the less permeable the product will become.

Hence, water impermeable mixtures are:

- asphaltic concrete;
- asphaltic mastic;
- dense stone asphalt;
- grouting mortar,

whereas water permeable mixtures are:

- lean sand asphalt and
- open stone asphalt.

Due to reasons explained in Chapter 9 of the Main Report only permeably bituminous mixtures will be considered in this Project.

2.7.2 Grouted Stones

Though there are different ways in which asphalt mortar can be used to grout stone, viz. fully grouting, pattern grouting or surface grouting, only the latter solution is considered in this case because of the necessity of water permeability of the revetments to be tested. In this type of grouting a certain quantity of grouting mortar is applied uniformly spread over the whole surface of the stone. The construction is not completely sealed since the mortar penetrates the surface layer only.

Grouting mortars are hot-type mixes of sand, filler and bitumen to which gravel can be added if necessary. The main advantage of this system is to have an open revetment which should, therefore, be combined with a filter construction on the soils. It can be used for grouting stone revetments above and below water level. Since the application of bitumen grout under water requires special equipment, techniques and much experience, this construction method will not be considered in this Project.

A revetment consisting of grouted stones is generally highly resistant to waves and currents. The stability is very high due to the stone skeleton and also the durability is good.

2.7.3 Open Stone Asphalt

Open stone asphalt is a mixture of bitumen and mineral aggregates with a gap in the mineral gradation between the sand and the crushed stone fractions. Usually the following mix compositions are applied:

- crushed stone : 80 %
- mastic : 20 %

with a composition of the mastic as follows:

- sand : 66.5 %
- filler : 16.5 %
- bitumen : 17.0 %

Open stone asphalt is an underfilled mixture used for revetments on slopes above water and as prefabricated mattress for slope and bottom protection also under water. Since it is permeable to water and sand, a filter should be applied in case of fine granular subsoil. The resistance to waves and currents is good, except in cases where gravel or heavier stones would regularly moved over the slope.

Open stone asphalt in situ must be placed above water and is stable on slopes of 1:1.5 to 1:1. When using prefabricated mattresses in which a filter cloth is already included, it can also be placed under water on slopes not steeper than 1:2.5.

2.7.4 Lean Sand Asphalt

Lean sand asphalt is an underfilled material consisting of sand and 3 to 5 % bitumen. The bitumen is acting as bonding agent for the mineral grains only and after some time the permeability is more or less equal to the sand of which it is made.

The material is used as filter layer, but also as a temporary or permanent revetment above and under water. In such cases a bitumen content of 4 % is usually applied. The material is, however, less resistant to waves and currents than open stone asphalt although the practice has shown a rather good performance as a temporary but as a also permanent revetment.

2.8 WIRE MESH

High quality galvanized or PVC-coated steel wire mesh is not produced in Bangladesh and has therefore to be imported. Only products of rather basic quality are available on the local market.

2.9 GEOTEXTILES

Geotextiles are a family of materials used in geotechnical engineering. The term geotextile defines both geonaturals and geosynthetics. The properties of geotextiles used with soils are to provide functions of reinforcement, filtration, drainage, separation, containment or protection or a combination of these functions. Their paramount advantage is that they are able to deform with the structure and still remain intact and operative if properly designed. When incorporated in bank revetments, geotextiles are mainly used as filters with drainage and separation functions. They can also be used as protective layers for example as a component of fabric mattresses.

The use of a reliable filter is an essential requirement for a sustainable revetment design and particularly for the underwater sections of revetments. Geotextiles in the form of geosynthetics are at present the only practicable materials for this purpose. The specifications of geotextiles have to meet both strength and filter requirements taking into account the factors resulting from variable loads acting on the bank protection, notably

- hydraulic loads;
- mechanical loads, and
- chemical and biological effects.

For further details as to the design and the specification of geotextile filters reference is made to ANNEX 17.

Geotextiles and related products can be classed as one-, two- or three- dimensional products. One-dimensional products include cables and strips and are mainly used for reinforcement. Two-dimensional products are sheet-like fabric materials which are commonly used as filters in bank revetments. Three-dimensional products such as bags, tubes, filament mats and mattresses can be used for reinforcement, containment and protection.

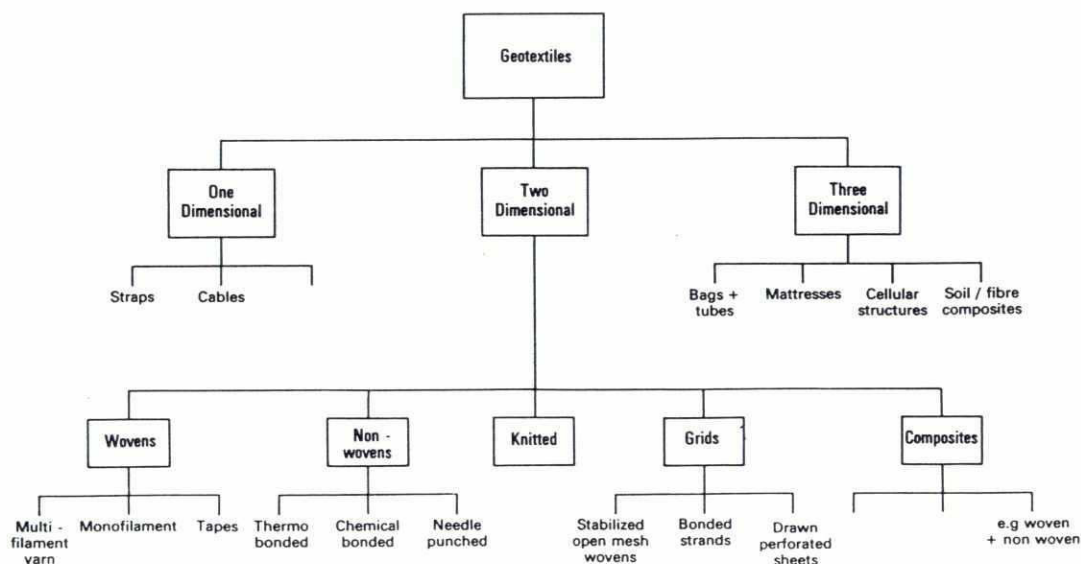


Fig.2.9-1: Type of geotextiles

(Source: PIANC, 1987a)

The more important products with regard to bank protection works are the two- and three-dimensional products. Within the group of two-dimensional geotextiles the wovens and non-wovens are of special importance. They have to comply with two hydraulic properties: permeability and filtration characteristics.

Geotextiles are usually materials of high permeability which have to allow water flow normal to the fabric and in the plane of the geotextile. The filter system must remain permeable in the long term whilst retaining soil particles. Thus the permeability criteria and the retention

criteria are decisive factors which are to be observed for designing geotextile filters. Further, geotextile filters must have adequate mechanical properties, such as resistance to tensile load and concentrated stress which can occur during construction of revetments. Finally, the material has to meet the requirements of durability according to their application. Whereas a filter in a permanent structure needs to be in working condition for the total life time of the structure, for other applications also less durable material may be used, particularly for temporary works or for works which are later protected by longterm filter.

In Bangladesh, geosynthetics are not produced until now although the scale of works in the field of filtration and retention of fine graded soils will in future require major quantities not only for hydraulic, but also for road engineering.

Only geonaturals such as jute sand bags are readily available in the country and are relatively low-priced. They are, however, presently only suitable for certain forms of temporary works, since they do not have sufficient filter properties and are significantly limited by their low durability. In a seminar on filter technology held by the Consultant some prospects for a combination of synthetics and jute arose, the geosynthetics taking over the permanent (filter) functions and the geonaturals the temporary functions, such as additional strength during the operation of placing. During the Test and Implementation Phase the Consultant will further look into the possibility of producing that type of geotextiles in Bangladesh.

At present however, the use of imported geotextile cloth, bags, tubes etc. is always necessary when a permanent filter function is required, as well as strength, flexibility, durability and sufficient resistance to puncturing/tearing allowing rough handling by mechanical equipment.

2.10 STEEL

There are several steel mills in Bangladesh producing reinforcement steel of different qualities, the raw material of which is, however, imported. Available quantities and qualities are sufficient for the proposed test structures.

Structural steel profiles in the form of beams, angles, girders, channels etc. are manufactured in Bangladesh with small dimensions only. Angles of maximum size of 10 x 10 cm are available on the local market. In case larger dimensions are required they have to be imported. Plane sheets of different sizes and thicknesses are produced in local steel mills and available in sufficient quantity.

Moreover, steel piles are manufactured at several places in Bangladesh. The required raw material ("hot rolled steel plates") is either imported or locally manufactured (Chittagong Steel Mill). The total annual production capacity is far higher than the actual demand in Bangladesh, however, steel is not produced on stock but only on demand, the delivery period reported to be 3 months.

3 CONSTRUCTION METHODS

3.1 GENERAL

The construction methods mainly depend on the criteria "what to build" and "where to build". It is a matter of fact that the infrastructure is poor at many locations near the Jamuna river. In many cases, access by road to the river bank is not possible or only after repair and reconstruction of existing roads.

After considerable parts of the embankment in the Kamarjani area were eroded during the monsoon season 1992, trucks have no more access to the selected test site. Several kilometers of construction and reconstruction of roads would be required. At the other test site at Bahadurabad the situation is similar. Some existing unmetalled roads in that area do not allow heavy traffic by trucks.

Therefore, it is planned to transport all the material and equipment needed for the execution of works at Kamarjani and Bahadurabad by boats and barges to the respective sites. Loading could be done at Chilmari, Rasulpur or Bahadurabad ghat. Further, it should be borne in mind that also for the execution of works floating equipment is needed, for example for the construction of groynes consisting of piles, but also for placing geotextile filters and revetment cover layers.

3.2 SLOPE PREPARATION

The test structures at Bahadurabad, in the form of slope revetments with a maximum water depth of 5 and 10 m respectively below SLW, are projected to be built in stagnant water immediately behind the then existing bankline. For that purpose excavation of a trench by dredgers from the riverside will be required. The slope of the new revetment must be built partly above and partly below SLW which is subject to changes during the construction period. The material from below the SLW can be excavated/dredged either by landbased or pontoon-mounted excavators or by cutter suction dredger.

The dredged/excavated material can be used to form hydraulic fill to the slope line, for raising the flood plain at the size and construction of connecting embankments. It may also be pumped to a stockpile or transported by barges for reclamation work elsewhere.

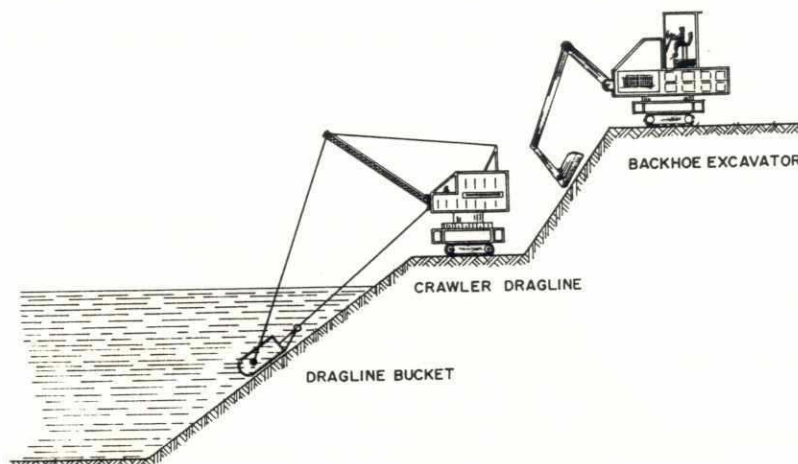


Fig.3.2-1: Excavation and slope profiling from the landside

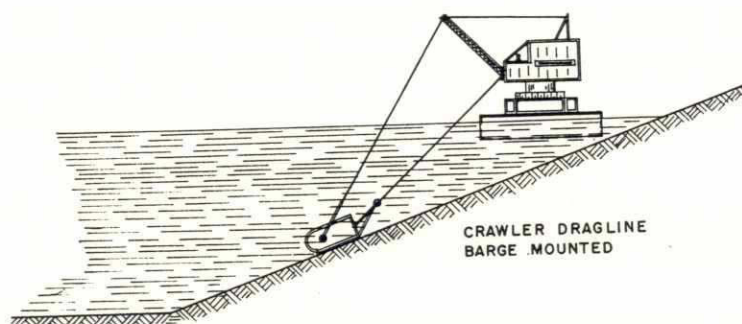


Fig.3.2-2: Excavation and slope profiling from the riverside

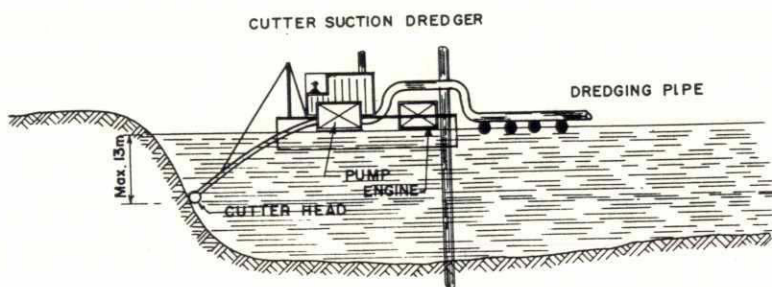


Fig.3.2-3: Excavation by means of cutter suction dredger

In case the underwater slope will not remain stable before placing the revetment, containment bunds are to be formed by dumping sand bags by a backhoe or crane. The outer slope of the bunds has to meet approximately the required slope line. The ponds so formed can then be filled by pumping in sand (see Fig.3.2-4).

Before placing revetment layers on the slope, the required slope line has to be checked and trimmed if necessary. Inaccuracies are to be removed either by excavator/dragline or by dumping/placing of sand filled geonatural or geosynthetic bags. The bags should not be

overfilled and it may be better to place them by crane or hydraulic arm. In any case it must be made sure that the bags do not burst during placing and that they form a well consolidated mass. The upper slope above SLW can be trimmed by mechanical means or by hand. Particular care is required to ensure a proper execution of the berm in the transition zone between the upper and the lower slope protection.

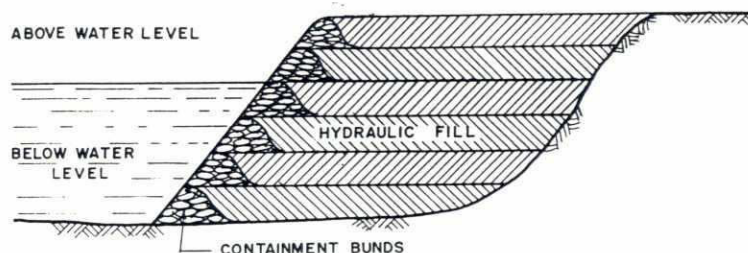


Fig.3.2-4: Slope preparation with the help of containment bunds and hydraulic fill

The finished slope surface must be sufficiently smooth to ensure that when placing the geotextile filter no deep voids would be covered. Such voids, if encountered, have to be filled with coarse granular material before placing the filter. Finally, the surface is to be checked by diver.

3.3 PLACING OF GEOTEXTILE FILTERS

The correct placing of geotextiles above water level does not offer significant problems and has become a more or less routine operation to experienced contractors.

Also if geotextiles are being placed under water, the fabric must be correctly positioned, free of folds and with proper overlapping, but this task is much more difficult to achieve than in the dry. Also under flow attack it must be kept in position until the cover layer has been placed, maintaining at the same time the lap zone free of ballast material. The fabric must be kept stable until the overlaying sheet is in position, and both the sheets are to be kept free of folds prior to placing the cover layer material. During placing the cover layer any puncturing of the geotextile has to be strictly avoided.

In order to fulfil the above mentioned requirements, a geotextile system must have both an appropriate tensile strength and an adequate flexibility to facilitate the handling. Furthermore, the complete system has to be sufficiently dense to remain in close contact with the subsoil while armouring is in progress.

There are several methods of laying geotextiles under water. The first approach does not require sophisticated equipment. The geotextile is unrolled from the landside, fixed to locally manufactured bamboo or fascine mattresses and pulled by a barge or pontoon to the riverside so that the system sinks under its own weight or after placing ballast. In a second method the

geotextile is unrolled under water from a large drum sunk to the toe of the slope. That drum is pulled up the slope after fixing the bottom end of the geotextile on the ground by ballasting it with armour material. The same applies for the geotextile unrolled on the slope which has to be continuously ballasted with armour material. It would be further advisable to extend the geotextile about 2 m under the launching apron and to dump part of the apron material on it to anchor the bottom of the sheets to assure a flow resistant transition from the toe protection to the slope protection. In dumping the protecting armour material from a pontoon or barge, the lap zone must be kept free of material until the next geotextile has been placed. If the cover layer consists of other than loose material, it might be advisable or even necessary to fix the geotextile filter to the armour material and to place both as one unit on the prepared slope. The third method is similar to the previous one with the difference that the drum with the geotextile is rolled down the slope. That method is however disadvantageous, particularly under flowing water conditions, since the armour material which has to be dumped continuously for ballasting the geotextile and fixing it to the slope will not stay stable. Due to the missing toe it would slide into the unrolling drum.

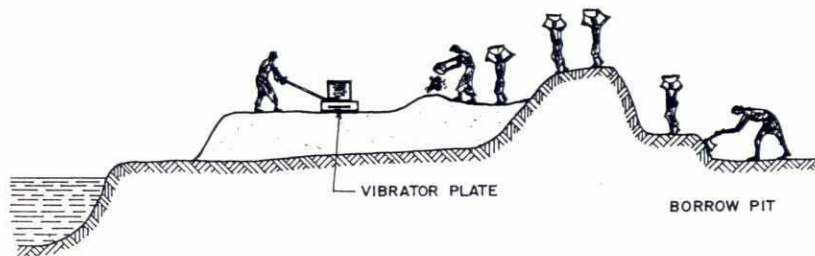
In any case it must be ensured that the geotextiles are properly fixed and anchored at the river bed and on the berm/top of embankment.

The lapping difficulty is common to all methods of placing geotextiles. The manufacturer of geotextiles should advise on the required lap width which should, however, not be less than 1.5 m in case of water depths up to 10 m at the time of placing.

3.4 EARTH WORKS

Apart from the works which are necessary to prepare the slope of revetments, including the possibly necessary containment bunds, other earth works are required to construct the embankment above the water level, the road on it and its land side slope. Furthermore, this type of work is needed for the construction of the impermeable part of the groynes in the form of earth dams. The fill material can be taken from the river bed or from the trench excavated in front of the revetments to be constructed. Above water it shall be placed in layers and compacted by mechanical means to avoid later settlements of the fill due to consolidation. The type of compactor depends on the type of soil and the layer thickness. If the dredged soil proves not to be suitable to build the required slope, containment bunds have to be formed also in that case to fill the created pond with dredged material (see Fig.3.4-2). Relevant requirements with regard to compaction of the fill must be observed. The sheet pile cofferdams can be filled with hydraulic fill which normally does not require additional compaction.

(a) by hand



(b) mechanically

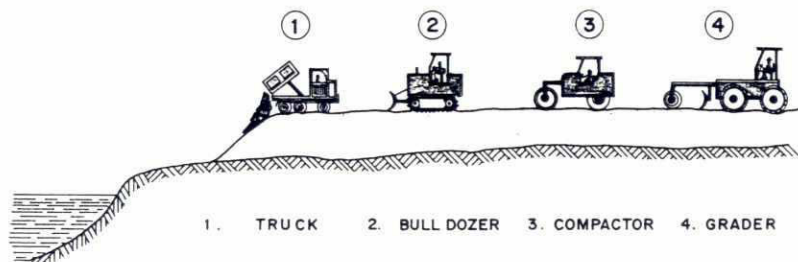


Fig.3.4-1: Construction of earth dams

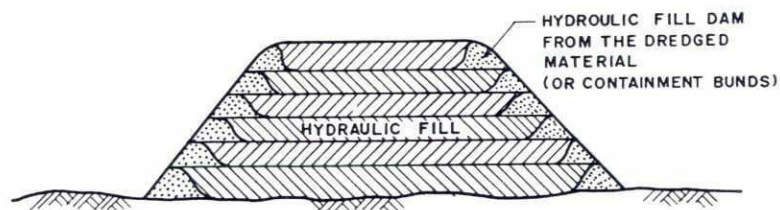


Fig.3.4-2: Construction of earth dam using hydraulic fill

3.5 PLACING OF ARMOUR LAYER MATERIAL

3.5.1 Loose Material

Boulders, and chemically bonded sand blocks for the construction of rip-rap as well as concrete blocks that are applied in the part above the waterline can be placed by hand or, in case the weight of the material required to resist wave and current attack is too high for manual handling, by means of specially developed clamps. With proper control, it is also possible to place rip-rap or concrete blocks under water. When using mechanical equipment it must always be ensured that the material is released as closely as possible to its final position.

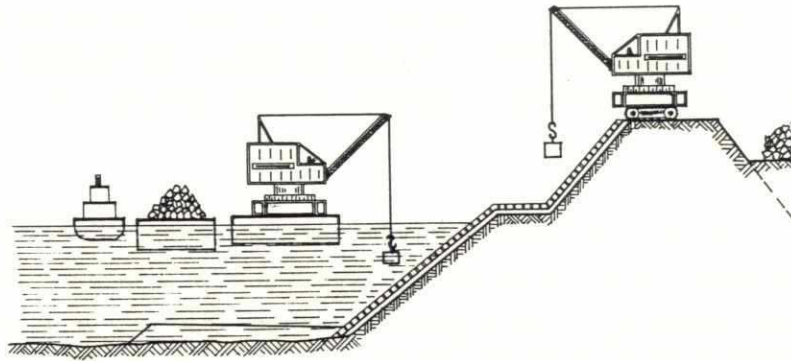


Fig.3.5-1: Placing of blocks from the landside or riverside

3.5.2 Cable and Geotextile Connected Blocks

The first system consists of concrete blocks linked by means of steel cables or synthetic ropes and cross wires forming a mattress of blocks, the size of which would be determined considering the hydraulic loads. In the second case the blocks are glued or bound to the underlying geotextile or fixed to it by other appropriate means.

In order to avoid high transportation costs the factory of these systems should be near to the site. Nevertheless, the selection of the fixed elements must allow for easy handling of the mattresses. Laying of the geotextile and the cable-connected block mats on top of it can easily be carried out separately, especially in the dry. The geotextile can be unrolled from the crest down the slope, followed by ballasting with the block mat. The same procedure can be continued on the underwater slope, provided that correct placing and overlapping are being controlled. The block mattresses must be anchored at the crest of the revetments.

Placing of the mattresses can be executed either from ashore or from a barge. For use from the waterside both the floating drum system as well as the application of a mat laying barge with a sloping deck would be possible. When using a crane with spreader bar, the equipment can be either land or pontoon based.

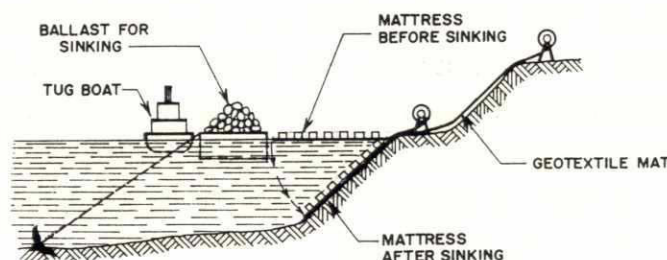


Fig.3.5-2: Placing of mattresses and geotextiles with ballast

The floating drum system can only be applied if the blocks are fixed to the geotextile. The mattress is rolled on a floating drum, transported to and installed at a pontoon, from where the mattress falls into place by unwinding the drum. However, the required movements of the anchored pontoon must be carefully controlled in order to guarantee an accurate placing of the mats.

The mat laying barge with sloping deck can only be used for the protection of the underwater slope. The barge is loaded with a number of mattresses ready for launching, which however, have a limited length depending on the length of the barge. If greater lengths are required, extension and completion of the mattresses can be done on the deck of the barge. After the laying barge has been brought into correct position, the top of the mattress is fixed to the river bank by means of cables or anchoring rods. When the barge moves out, the mattress falls into place along the sloped bank.

If a crane, either landside or pontoon based, is equipped with a spreader bar, the mattresses can be lowered down to the slope of the bank, where the spreader bar is disconnected mechanically and after that ready for the next mattress. Careful positioning is also indispensable in this case. The maximum size of the mattresses depend on the capacity of the crane and on the transport system for the prefabricated mattresses.

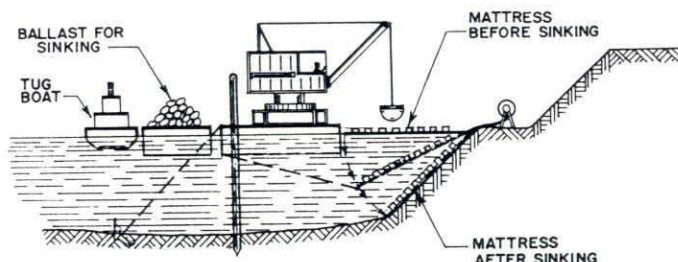


Fig.3.5-3: Placing of mattresses

3.5.3 Bitumen Grouted Stones

This method should be applied only above the water level. Details are given in Subsection 2.7.2 of this annex and Subsection 9.2.2 of the Main Report. The asphalt grout can be mixed in normal mixing plants. Only in cases where very coarse aggregate is added it may be necessary to use special mixing equipment. The mixing temperature lies between 130° and 190° Celsius. The added bitumen should be just sufficient to cover all grains of the aggregate with a thin coat. The hot mortar can be applied using buckets or directly from a asphalt container. However, it must be ensured that only surface grouting is executed without reducing the permeability of the cover layer.

3.5.4 Cement Grouted Stones

Also grouting with cement mortar should be executed only above the water level. The required equipment is a normal mixing plant. Reduction of the permeability is to be avoided and therefore only surface grouting should be allowed, reducing the nominal size of stones required for stability under wave and current attack.

3.5.5 Lean Sand Asphalt

Lean sand asphalt can be produced in a normal batch or drum-mixer. The mixing temperature with a batch-mixer is between 125 to 190° Celsius the exact temperature depending on the required viscosity and the transport distance. The material must be placed in one operation to the correct thickness.

3.5.6 Wire Mesh Gabions

Gabions are rectangular cages made of hexagonal double-twisted wire mesh and filled with appropriately sized stones, boulders or bricks. Various mesh sizes and box shapes of different dimensions are available. The wire is made of soft galvanized or PVC coated steel. Gabions are normally about 1.0 m high and 0.5 to 1.0 m wide. In case they are only about 0.2 m thick, they are called mattresses. Mattresses of larger sizes can be subdivided into approximately equal sized cells. Filling of gabions and mattresses can be carried out manually or with ordinary mechanical equipment. If possible, filling should be done preferably at the final spots on the site, since otherwise the transport of the filled gabions/mattresses from the filling station to the final location may cause deformation or rupture of wires. To ensure best performance, properly sized filling material should be specified avoiding flat stones. The units should be filled tightly to prevent movement of the stones.

The advantage of gabion structures is that they can be manufactured without heavy equipment. Before placing the units, a geotextile is to be placed on top of the subsoil. Closing and horizontal linkage of individual mattresses is a manual operation if it can be done in the dry. Placing of mattresses on an underwater slope can be executed with sloping deck barges, where filling and linking together of units to larger dimensions can be carried out before the sinking procedure starts. Provision has to be taken to transport the fill material to the manufacturing and placing barge.

3.5.7 Fabric Containers

Fabric containers may be either bags or mats or tubes which are produced from geotextiles or other materials. Bags can be filled with sand and directly placed on the prepared slope, above water by hand and below water by means of cranes only. They should not be dumped

in order to avoid their bursting, but released as closely as possible to its final position. A filter layer is required if bags are placed one layer thick only. Otherwise at least two layers of bags should be placed to avoid loss of core material. The bags shall not be filled completely but only to about 70 % to provide for a better adaptation to the sublayer due to their flatter shape a better flexibility. Unless covered by a durable filter and coverlayer material, the bags must be made of geosynthetics to ensure the required longevity.

Other fabric containers such as tubes and mats are designed to be placed directly on the prepared slope. Normally, they are joined together, placed empty and then filled with sand, lean sand asphalt or concrete. This results in a mass of pillow-like units with regularly spaced filtermeshes for hydrostatic pressure relief.

Since geosynthetics are not produced in Bangladesh until now, importing of that material will be necessary. Cutting and sewing to the correct dimensions has to be carried out by trained labour. The filling method can be either mechanical, hydraulical or pneumatical.

Mats of larger dimensions can also be filled ashore on top of the slope above water. The filled parts would be pulled into the water using a recoverable sinking beam which is linked to the lower edge of the filled mat. The pulling winches are installed on a positioning pontoon. Placing can also be carried out by means of a crane with spreader bar, by the rolling system or with a sloping deck barge. Anchoring of the mats on top of the slope is important.

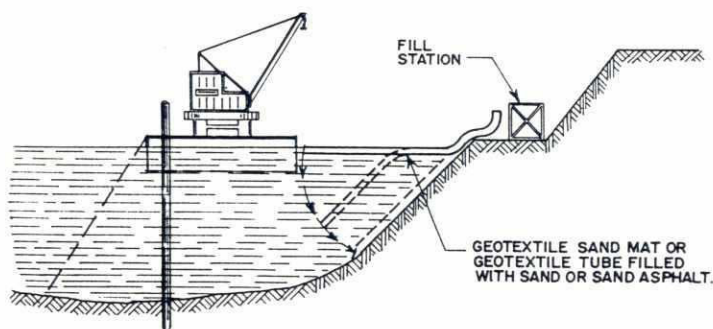


Fig.3.5-4: Filling and placing of fabric containers

3.5.8 Toe Protection

The protection of the toe of a slope revetment is carried out preferably by dumping loose material to the river bed in sufficient quantity, allowing the material to adjust to the scour hole.

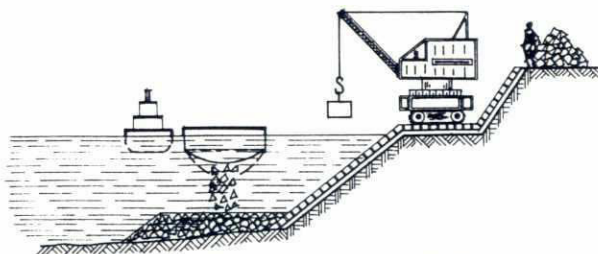


Fig.3.5-5: Construction of falling apron

The same result can be achieved by block mats or tubular fabrics, which have, however, to be placed prior to the slope protection. Generally, both components would be placed by means of the same equipment. Geotextile bags should be not filled completely for the same reasons as explained above.

Scour prevention mats may be completely prepared ashore and then placed by a sloping deck barge or by a crane with spreader bar.

3.5.9 Sinking Piles

The groynes at Kamarjani test site are designed to be partially impermeable and partially permeable. The latter part would be constructed of steel piles and concrete piles. Sinking of piles must carefully be planned with regard to the prevailing subsoil conditions. Driving might be problematic as may be the method of vibration. Most probably the piles must be sunk by means of water jetting. The sheet piles of the cofferdams can probably be driven relatively easily due to their rather short length. In any case suitable pile driving equipment must be available on site as well as a pontoon to carry the crane/pile driver.

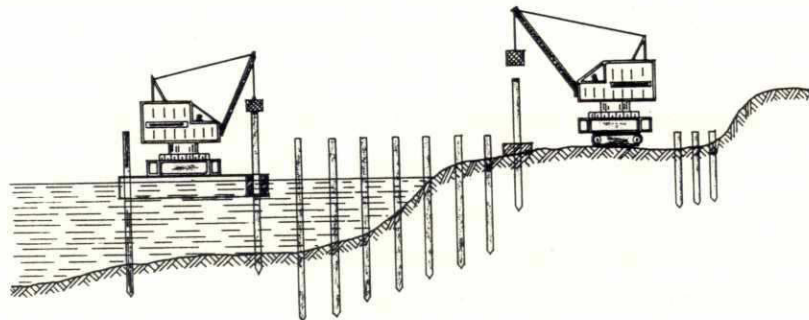


Fig.3.5-6: Pile driving on landside and from floating pontoon

4 CONSTRUCTION EQUIPMENT

The explanations in the subsections above have shown that for the execution of works at both test sites not only the usually needed construction equipment is required but also heavy and sophisticated equipment, such as sloping deck barges, cranes with spreader bars, pile driving equipment, cranes of sufficient capacity, pontoons etc. for special works. It should be borne in mind that manual work or basic mechanical equipment is not sufficient to carry out any technically sound under water bank protection works.

In Bangladesh there are two groups of owners of construction equipment notably

- Government owned organizations and
- private contractors.

Organizations of the first group are:

- Bangladesh Water Development Board (BWDB);
- Bangladesh Inland Waterways Transportation Corporation (BIWTC);
- Bangladesh Inland Waterways Transportation Authority (BIWTA), and
- Roads and Highways Department (R&HD).

The great number of private contractors in Bangladesh is mainly involved in construction of buildings and roads and the reconstruction works of the latter. In many cases they are only in possession of such equipment they need in their special field of work such as small mixers, and may be handhold compactors.

Another but smaller group of contractors, so-called class A contractors, are qualified to construct larger buildings, bridges, regulators, pumping stations, bank revetments etc. They have more or less their own equipment like concrete mixers, small cranes, trucks, small crushers, asphalt mixers, generators. Only a few contractors are in a position, with regard to their equipment, to build more important constructions for which bulldozers, graders, cranes of sufficient capacity, heavy trucks, crawler cranes, excavators and floating equipment are required. However, it is common practice in Bangladesh that also these contractors are hiring the equipment needed for the execution of contract works.

The Government organizations can present a long list of equipment. Recent investigations by the Consultant give rise to the assumption that this equipment may often, however, be in a rather poor condition due to lack of funds for repair and maintenance. Sometimes it is even not worth to be repaired under economic terms. Therefore, the equipment is occasionally offered to private contractors on condition that the latter will execute necessary repair works, expenses for which will be deducted from the lending fees. To have a sound assessment for the construction phase, particularly to prepare hire (or purchase) of equipment, special and detailed investigations are anticipated by the Consultant during the Test and Implementation Phase and which will be undertaken by an experienced mechanical engineer, specialized on construction equipment.

BWDB is the largest of the Government organizations and among other things owner of 27 cutter section dredgers out of which about 50 % seem to be in working condition. They can be hired on fixed rates per m³ dredged material. Other heavy equipment like tug boats, barges, pontoons, bulldozers, crawler draglines, motorgraders, dump trucks and others can be hired on daily or monthly rate basis according to a published "Schedule of Rates".

However, no cranes nor excavators are available at present. According to informations obtained from the Department of Mechanical Equipment of BWDB, 33 new backhoe - excavators of different capacity and 6 new mobile cranes with a capacity of 25 tons and with hydraulic telescope beam are expected to be delivered soon.

In this connection it is to be mentioned that a considerable number of crawler draglines which could be used as multipurpose equipment in bank protection works were found parked at Teesta Barrage in August 1992. BIWTC and BIWTA were found to have a limited number of equipment being, however, almost full time engaged on their own works.

The Roads and Highway Department operates in the same way as BWDB, but their equipment is of course more specialized in the field of road construction.

Most of the equipment needed for the construction works at the test sites of FAP 21 is available in Bangladesh, however, in some cases repair works are urgently required in order to put it in working condition.

Moreover, for some special works a versatile multipurpose crawler crane needs most likely to be imported as the key equipment for the construction and adaptation works on both test sites. Other equipment would additionally be hired from GoB organizations or other contractors. Presently, the Consultant would tentatively specify the equipment to be purchased as follows:

Crawler crane with about 100 ton capacity with the following appliances:

- second (+ third ?) drum for dragline operation
- clampshell
- orange peel shell for placement of boulders
- comprehensive pile driving equipment including
 - . a (diesel) hammer with variable blowing energy
 - . a vibrator with adjustable frequency and energy
 - . a sufficiently strong jetting device.

This rather comprehensive equipment will be required during the test phase, firstly to find the optimum driving system and secondly to optimize that system as to power, frequency, discharge or the like.

For future (standard) execution it would be sufficient to utilize a more straight forward equipment according to the optimized specifications. Similar considerations will be true for specifying the pontoon to carry the crane/pile driving equipment.

5 ASSESSMENT OF CONTRACTORS' CAPABILITIES

One of the objectives of this study was to obtain a comprehensive knowledge of local contractors, their organization, staff, equipment and especially their competence, qualification and experience in the field of earthworks, bank protection and river training.

According to the informations the Consultant collected during the Planning Study Phase, there are several thousands of contractors of all magnitudes with regard to their manpower and equipment registered in Bangladesh. The majority of them however, are relatively small companies which do employ staff but hire equipment according to demand. This method is also applied even by larger companies which only own the basic equipment that is regularly used in their field of works. For all special works, equipment is hired from Government organizations or other parties as explained in the previous subsection.

Contractors working for GoB organizations are registered in classes. The highest ranking class is the first or class "A". For example, about 500 class "A" contractors were registered with the Bangladesh Water Development Board (BWDB) for the financial year 1990/91.

Criteria for enlistment of "A" class contractors are as follows:

- i) Experience of the firm in execution of the following works:
 - earthwork in construction of flood embankments, cross-dams and road embankments, irrigation canals;
 - earthwork in excavation and re-excavation of drainage channels;
 - construction of hydraulic structures, and
 - river bank protective work.
- ii) Volume and cost of works executed by the firm during three years preceding the year for which enlistment is applied for (required to be certified by the concerned Executive Engineer).
- iii) Strength of technical personnel (graduate civil engineer or diploma civil engineer with minimum of 3 years' experience) of firm. A firm must have at least one engineer to be qualified for enlistment.
- iv) Financial solvency of firm (certificate from a bank regarding solvency along with bank transactions statement for preceding three years in support of solvency, are required to be submitted with application for enlistment).
- v) Strength of equipment and machinery owned by firm for execution of the works mentioned under i) above (a statement showing type, number, make, capacity of equipment and machinery along with a certificate from an Executive Engineer of BWDB in support of the said statement, is required to be submitted with the application for enlistment).
- vi) Favourable recommendation of the Chief Engineer of the BWDB through whom the application for enlistment is submitted to the competent authority.

The extent of work experience, the volume and cost of work executed during the period as mentioned under iii) above, the amount of money involved in the Bank transactions as mentioned at iv) above, the strength of equipment and machinery as mentioned under v) above, and the amount of Bank balance which should be considered adequate for enlistment of "A" class contractor is left at the judgement and discretion of the competent authority of the BWDB (Member, Implementation).

In order to get an idea of the experience, equipment and capability of local contractors on the field of larger bank protection works, the Consultant addressed a questionnaire to about 50 class "A" contractors with the request to submit their documents on the following items:

1. name, address, status of company;
2. managing and leading staff information including responsible engineers;
3. list if equipment (brand, year of manufacturing, capacity, condition a.s.o) owned by the company and related to the above mentioned works;
4. financial situation, bank solvency certificate, and
5. references for projects executed during the last 10 years, related in bank protection or similar works of approx. same (or more) amount as required for the Project, including previous joint venture experiences or collaboration.

Thirty-three answers were received and a detailed evaluation of the documents shows that quite a number of contractors, partially with some reservations, seem to be capable to execute the construction works for the test structures at Kamarjani and Bahadurabad. The reservations apply to the practical experience in large-scale hydraulic works such as proper grading of underwater slopes, accurate placing of large size mattresses under water, availability of pontoons and other heavy or specific equipment.

To overcome these difficulties of insufficient experience in the type of works to be executed for the test structures it is recommended, with special regard to the tight time frame of about 8 months for each site, that the contractors finally prequalified may engage foreign specialists or may obtain advice from foreign firms well experienced in bank protection works.

Some of the Contractors who submitted their documents entered into a joint venture with foreign companies from India or China in which the foreign partner is generally the more experienced one with substantial equipment for all types of work. With regard to the required experience and equipment which is indispensable under all circumstances, such a joint venture might be of interest and a reasonable option for the execution of the test structures.

6 CONCLUSIONS

Most of the materials needed for the construction of test structures at Kamarjani and Bahadurabad are available in Bangladesh. Only geosynthetics for the construction of revetment filter layers and some steel products such as sheet piles are to be imported. Steel sheet for rolling of piles may be imported by the Consultant for expediting manufacturing or for reducing costs.

With regard to the partially new construction methods and materials which require specific experience and partly special equipment the assistance of competent local contractors by foreign experts or even foreign companies in the form of joint ventures is recommended.

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(FAP 9B)

Meghna River Bank Protection Study,
Final Report (and preceding reports)

RPT, NEDECO, BCL, (1988)

Jamuna Multi-Purpose Bridge Study

**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 20

**STUDY OF ALTERNATIVE DESIGN
SOLUTIONS**

JANUARY 1993

STUDY OF ALTERNATIVE DESIGN SOLUTIONS

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1 SELECTION OF FEASIBLE TEST MEASURES

1.1 PRESELECTION

In Section 4.2 of the Main Report all types of measures and structures have been explained that are of permanent nature and suitable for bank protection in braiding rivers. Out of those cited, some may be disregarded for test structures right from the beginning for various reasons, viz.:

- Retaining structures,
being expensive in consideration of the scour depths expected. Such structures might only be required for loading/unloading of bigger ships.
- Guide banks,
being expensive due to their forward position and required for important infrastructure etc. only. In fact, due to required termination points at up and downstream ends of revetments, these get a similar shape as guide banks.
- Training walls,
being expensive due to their continuous construction within the channel and in the first instance required for navigational purpose.
- Sills and closure dams,
being expensive and serving special purpose, not considered for standard application.
- Capital dredging,
can also not be a standard solution for bank protection, being suitable for favourable planform locations only.

Therewith remain the following structures which will be looked into in more detail:

- 1- Slope revetments
- 2- Groynes, including
 - a- solid groynes;
 - b- permeable groynes, and
 - c- bank heads, which may be considered as short groynes in this context.

Advantages and disadvantages of each of the above structures will be highlighted and assessed at first, to acquire comparable criteria thereafter.

1.2 ADVANTAGES AND DISADVANTAGES

1.2.1 Slope Revetments

Advantages over Transverse Structures:

- construction can be phased to a certain extent in accordance with the budget available, if provision against outflanking is made;
- if properly designed structure is equally suitable after change of flow (direction of attack);
- structure does not obstruct navigation;
- structure has no or little adverse effect on other river parts;
- structure causes less disturbance to river flow than transverse structures, resulting in less scour depth and current velocity increase (75% for rough estimation), and
- structures for greater depth (e.g. in concave bends with active erosion) are less costly.

Disadvantages:

- structure is exposed to current and waves over its full length;
- construction requires bank clearing and grading for developing a proper slope, using considerable land area, and
- structures would become superfluous if the thalweg of the river recedes from the bank line (no adaptation to new situation possible).

1.2.2 Impermeable Groynes

Advantages over slope revetments:

- construction cost may be less if the structures due to local conditions can be spaced wide enough apart and need not to be extended into deep channels;
- structures can be adapted to changing river conditions (length, head, shape);
- structures can be erected somewhere upstream to repel or direct the flow, rather than to protect the place of deep scour directly;
- structures are built into the river rather than to occupy valuable land;
- heavy protection is required only along the heads and at the upstream face;
- set of groynes is suited for channel correction (navigation) and long term river strategies (river stabilization), and
- Siltation in front of existing bank line can be induced in a groyne field.

Disadvantages:

- structures represent disturbance and danger points for navigation;
- structures due to their obstruction to the current flow create eddies and thus deep scours (approx. up to 1.4 times of those for slope revetments);
- structures, therefore, are maintenance expensive and usually require constant repairs;
- construction cost rises extensively if to be erected in deep water;

- construction must be completed during "construction windows" (low water periods) to avoid complete failure;
- structures are only effective in series, not as single groynes;
- structures are vulnerable to embankments and flank attack, and
- structure influences other locations (area of attack shifted downstream or to opposite banks).

1.2.3 Permeable Groynes (permanent structures)

Advantages:

- structures combine most of the advantages of solid groynes but may be spaced even wider apart;
- flow of the channel/river does not change abruptly, thus eddies will be weaker and scouring less, and
- part construction during one season is possible without causing complete failure, but construction may even continue during moderate flood.

Disadvantages:

- structures are only fully effective in aggrading river type with high bedload and suspended load;
- structures may be vulnerable against shock and pressure (floating logs, water hyacinths, ship impact) and human mischief;
- design of spacing/pillar relation is not yet quantifiable due to lack of empirical and model test data, and
- construction and maintenance requires higher technology.

1.2.4 Bank Heads

Bank heads act either as a combination of slope revetment and short solid groynes or (usually) are designed to reduce the length of revetments.

1.3 MULTI-CRITERIA ANALYSIS

In the following a multi-criteria analysis (MCA) will be carried out to find the order of preference for test structures. For this purpose first and second order criteria have been selected.

The 1st order criteria with their weighing factors, specially applicable to test structures, are as follows:

- Functional Criterion : 25%,
expressing the ability to reach the desired aim

- Suitability Criterion : 25%,
including both the universality of the structures for standard design but also maintenance features, directly related to testing. Durability (life time) of the structures, usually an important part of maintenance is, therefore, included in the functional criterion
- Construction Criterion : 20%,
reflecting the problems to be faced during erection
- Flexibility Criterion : 15%,
expressing the ability to react to circumstances changed after erection
- Environment Criterion : 15%,
including effects on environment and human factors

In Table 1.3-1 the above listed main criteria (I) will be detailed by 2nd order criteria (II), each given a certain percentage share (total 100%) according to their importance.

In Table 1.3-2 these criteria will be applied to each alternative, thereby giving the following scores S = 2, if answer to statement in the second order criterion is yes
= 1, if answer is indifferent
= 0, if answer is no.

The second column of each alternative gives the number of points P, calculated as follows:
$$P = I (\%) * II (\%) * S * 100$$

These will be added to derive at the total number of points per alternative in the last row of Table 1.3-2.

Main Criteria	Share I (%)	Second Order Criteria	Share II (%)
Functionality	25	Deep water suitability	20
		Bankline stabilization	20
		Durability	20
		Siltation promoting	15
		Channel stabilization for navigation	10
		Resistance to human mischief	10
		Boat traffic supporting, no danger to navigation	5
			100
Suitability	25	Monitoring of all parts easy	25
		Number of alternatives for test high	25
		Universal applicability	20
		Repairs can be carried out easily	20
		Flexibility in design possible	10
			100
Construction	20	Short time required for achieving stable unit/section	25
		Mechanization required is low	25
		Quality control is easy	25
		High current condition presents no problem	25
			100
Flexibility	15	Adaptable to increased depth (scour)	30
		Stability during earthquake	30
		Adjustable to changing river conditions	25
		Adaptable to subsoil settlements	15
			100
Environment	15	Low land requirement	25
		Low danger level in case of failure	25
		Low economic consequences in case of failure	20
		Social factors positive	15
		Positive effect on fishery	10
		Landscape improved	5
			100

Table 1.3-1: Criteria for test structure selection

Criteria				Test Structure Alternatives							
Main		Second order		Slope Revetment		Solid Groynes		Permeable Groynes		Bank Heads	
	I(%)		II(%)	S	P	S	P	S	P	S	P
Function- ality	25	Deep water	20	2	10.0	1	5.0	0	0	1	5.0
		Bankline stabilization	20	2	10.0	2	10.0	2	10.0	2	10.0
		Durability	20	2	10.0	1	5.0	0	0	1	5.0
		Siltation	15	0	0	1	3.8	2	7.5	0	0
		Channel stabilization	10	1	2.5	2	5.0	2	5.0	1	2.5
		Resistance	10	1	2.5	2	5.0	0	0	1	2.5
		Boat traffic	5	0	0	1	1.3	2	2.5	0	0
Suitabi-lity	25	Monitoring	25	2	12.5	1	6.3	2	12.5	2	12.5
		Alternatives	25	2	12.5	0	0	0	0	1	12.5
		Universality	20	2	10.0	1	5.0	1	5.0	2	10.0
		Repairs	20	2	10.0	1	5.0	0	0	2	10.0
		Flexible design	10	2	5.0	1	2.5	1	2.5	1	2.5
Construc- tion	20	Time required	25	2	10.0	0	0	2	10.0	1	5.0
		Mechanization	25	2	10.0	1	5.0	1	5.0	1	5.0
		Quality control	25	1	5.0	1	5.0	2	10.0	1	5.0
		High Current condition	25	1	5.0	0	0	2	10.0	1	5.0
Flexibi-lity	15	Scour	30	1	4.5	1	4.5	1	4.5	1	4.5
		Earthquake	30	1	4.5	1	4.5	2	9.0	1	4.5
		River change	25	1	3.8	2	7.5	2	7.5	1	3.8
		Settlements	15	2	4.5	1	2.3	2	4.5	1	2.3
Environ- ment	15	Land required	25	0	0	2	7.5	2	7.5	1	3.8
		Danger level	25	2	7.5	1	3.8	2	7.5	1	3.8
		Econom. consequences	20	1	3.0	0	0	2	6.0	1	3.0
		Social factors	15	1	2.3	2	4.5	1	2.3	1	2.3
		Fishery	10	0	0	2	3.0	2	3.0	0	0
		Landscape	5	0	0	2	1.5	1	0.8	0	0
Total number of points					145.1		103.0		132.6		120.5
Ranking				1		4		2		3	

Table 1.3-2: Comparison of test structure alternatives

1.4 CONCLUSIONS

The result of the foregoing MCA clearly indicates that preference should be given to slope protections (revetments). In weighing the different criteria, however, it has to be kept in mind that:

- All scoring can only be relative, i.e. the ranking is a comparison of the different alternatives for similar situations, but does not give solid facts. For instance, mechanization "low" refers to the fact that the grade of mechanization (as percentage of to total cost) for a revetment work site is low in comparison to pile driving for a permeable groyne, but heavy equipment involved may be of the same size. Similarly,

"easy" adaptation to subsoil settlements for revetments in comparison to groynes depends actually on the type of armour layer used.

- Ranking as per Table 1.3-2 is not a rigid statement but has to be judged again for different local conditions and other circumstances. The advantages and disadvantages for each alternative as summarized in Section 1.2 have to be considered in final selection for a definite location.

Under consideration of the analysis carried out in above Section 1.3 and the sites selected in Section 8.2 of the Main Report it is proposed to test the following structures:

- 1 a combination of permeable and impermeable groynes at Kamarjani test site, and
- 2 slope protection with different revetments and toe protections at Bahadurabad test site.

2 ALTERNATIVE DESIGN

2.1 INTRODUCTION

Based on morphological studies several possible test areas have been investigated both on the right and on the left bank of the Jamuna (see Section 8.1 of the Main Report) out of which finally 2 sites have been selected for the construction of test structures (see Section 8.2 of the Main Report). The determination of the sites was obviously one of the key decisions of the Project: after all, the structures are not only to be tested but also to serve this protection purpose. To allow continuous discussion, checking and optimization in selecting the test sites a procedure was followed which started right at the beginning of the Project and consisted in a stepwise narrowing of the alternatives. A set of criteria was developed to check the alternative areas against. One of the main and decisive criterion was the "Certainty-of-Attack" Criterion which took into account the past and often erratic changes of the river's planform. Past morphological changes were evaluated and forecast methods were developed to determine with a certain degree of confidence that the preselected sites would remain exposed to flow attack throughout the Test and Implementation Phase. Not less important was the "Something-to-Defend" Criterion taking into consideration that the test structures should be erected at locations which require the protection of a town, infrastructure or the like making practical use of the investment and allowing the population the acceptance of the Project. Furthermore, the "Fitting-in-Overall-Strategy" Criterion served as a link between the two project components FAP 21 and FAP 22. It had to make sure that the hard points created by the Bank Protection (FAP 21) component would fit into the overall strategy of the River Training/AFPM (FAP 22) component.

The finally selected test areas are located at Kamarjani and Bahadurabad respectively (see Fig.A.20-1). At Kamarjani, it is planned to construct in 1993 a field of 5 groynes and at Bahadurabad one year later a number of slope revetments being different in composition,



dimensions, construction materials, construction methods and toe protection.

Kamarjani is situated on the right bank of the Jamuna, about 15 km downstream of the confluence of the Teesta river. The area experiences serious bank erosion which is likely to continue over the coming years. Hence the probability of the test structure to be exposed to flow attack until the end of the Test and Implementation Phase is rather high. The test site is situated upstream of the Manos regulator, an important infrastructure which would be protected as would be the village Rasulpur downstream of it. The test structure would finally be the northern most fixed point on the right bank between the confluences of the Teesta and the Ganges in a series of 8 to 10 "hard points" to prevent the right bank moving further to the west.

Bahadurabad is the only test area on the left bank promising to have a fair probability for a continuous and sufficient flow attack during the Test and Implementation Phase. Bahadurabad is the eastern end of the most important railway ferry connection of the country. The test structure would help stabilizing the terminal area to avoid too large reconstruction works of the station and the access tracks and at the same time protect the village of Belgacha from being washed away from continuous erosion.

2.2 TEST SITE KAMARJANI

2.2.1 General

The finally selected site within the test area of Kamarjani is situated about 3 Km north of the Manos regulator (see Fig.A.20-2). The change of the bankline during the monsoon season of 1992 as shown on Fig.A.20-2 elucidates that the river bank in that area is still undergoing heavy erosion which is expected to continue during the next forthcoming years with a probability of at least 67% (see Section 8.2.2 of the Main Report). Nevertheless, the total bank erosion will be checked again at the beginning of 1993 when the next satellite images become available.

Groynes provide an indirect protection to a river bank by either repelling or deflecting the current, or reducing the flow velocity and thus forcing the bed load to settle down and create protective sediments in front of the banks. The most important distinction of groynes is according to their structure, either solid (impermeable) or open (permeable). The latter usually consist of one or several rows of piles (steel, concrete, timber). Due to their spacing water can pass but current is slowed down causing deposition of sediments behind. Solid groynes are built from stone, gravel or rock, as earth dam with slope protection or as concrete structures.

Since there is no rule of general applicability for determining the plan view of groynes just as other dimensions depending on a number of different design parameters, in many cases physical model tests are required. One of the objectives to be achieved by FAP 21 is to

develop and optimize design criteria for different types of bank protection including groynes. The aim of the physical model investigations was to define the optimum alignment and layout of the test structures and to determine the flow velocities in their vicinity as well as the maximum scour hole depths and the location of the scour holes.

Since it is planned to construct at Kamarjani a set of groynes, the main aim of the model investigations was to test the effects of the main design parameters:

- the type of the groynes
- spacing of groynes
- the slope of the head of the groynes
- the length of the groynes
- the direction of current approach

and to provide the design criteria for the flow velocities in the vicinity of the groynes and the maximum scour depths.

One of the most important results of the investigations was to realize that the depths of scour holes at the head of permeable groynes is considerably less than that at solid groynes. This phenomenon could even be influenced by gradually increasing the spacing of piles of the permeable groynes. Based on the results of the physical model tests it was decided to construct a series of 5 groynes at Kamarjani. The total length of the piles will be 120 m with an orientation of 15° to the flow direction. the spacing was selected to be 250 m. It is intended to built them over a length of about 60 m on the flood plain as impermeable groynes in the form of conventional earth dams and as steel sheet pile coffer dams, whereas the other 60 m length will consist of pile structures using steel piles and concrete piles as well (see Fig.A.20-3). Since the scour depth to be expected at the groyne head is in the order of about 6 m only, the provision of a falling apron is not required. However, the scour depth has to be taken into account when determining the required embedded length of the piles.

2.2.2 Design Criteria

Based on the results of topographical survey, hydrographical survey, hydrometric measurements, physical and mathematical model tests the following design criteria for the test structures at Kamarjani were fixed:

(i) **water levels:**

- standard low water level : + 15.2 m PWD
- Standard high water level : + 20.7 m PWD
- high flood level : + 23.3 m PWD ($\hat{=}$ SLW + 8.1 m)
(100 years return period)
- design water level : SLW + 7.7 m
(25 years return period)

(ii) Soil conditions:

- flood plain level : + 20.5 to 21.0 m PWD
- based on the results of boring No. GA 2 and GA 4 the following soil properties can be assumed

γ/γ'	=	18/8 kN/m ³
φ'	=	27.5°
c'	=	0 kN/m ²
- relative density: down to about + 5 m PWD : loose
- below $\pm$ 0 m PWD : medium dense to dense
- groundwater level : about + 17.5 m PWD

(iii) hydraulic loads

- design velocity : $u = 2.5$ m/s
- significant wave height : $H_s = 1.0$ m
- wave period : $T = 3$ s.

(iv) Scour depth:

- at the head of permeable groynes : 6 m below river bed
- at the head of solid groynes : 20 to 30 m below river bed

The greater depths are to be expected at the first groyne of a series, the lower values at the following ones.

(v) Earthquake loads:

Earthquake loads and liquefaction of soil is not considered, since the frequency of occurrence is rather low and cannot be expected during the Project period. Hence the test structures would be "overdesigned" which is not according to the design philosophy that requires a lower level of safety to allow a certain damage of the structures.

2.2.3 Alternative Design Solutions

As mentioned in Subsection 2.2.1 groynes may be built

- either impermeable (solid)
- or permeable (open),

consisting of earth dams or sheet pile cofferdams in the former case and pile structures (steel or concrete) in the latter case. Since the physical model tests on groyne fields showed satisfactory results when the groynes were partially built on the flood plain, this solution was followed up and the best results were obtained when having about 60 m out of a total length of 120 m constructed on the flood plain.

Taking into account that the required crest height of the groyne is only about 2.0 m above the flood plain level, it seems to be reasonable also with regard to the flow pattern between the groynes, to built the part of the groynes on the flood plain as impermeable groyne in the form of earthdams or sheet pile cofferdams. The remaining length to be constructed in the river exposed to its current can be executed either as solid or open structure. Thus the following alternatives are feasible (see Fig.A.20-4 to A.20-9):

- Type GA-1: . impermeable
 . earthdam on the flood plain and in the river
- Type GA-2: . same as type GA-1, however, partially submerged at high water stage
- Type GA-3: . impermeable stretch on flood plain: earth dam
 . permeable stretch in the river: steel piles
- Type GA-4: . impermeable stretch on flood plain: earth dam
 . permeable stretch in the river: steel piles
 . partially submerged at high water stages
- Type GA-5: . same as type GA-3, however, concrete piles instead of steel piles
- Type GA-6: . same as type GA-4, however, concrete piles instead of steel piles
- Type GA-7: . impermeable
 . sheet pile cofferdam on the flood plain and in the river
- Type GA-8: . same as type GA-7, however, partially submerged at high water stages.
- Type GA-9: . impermeable stretch on flood plain: sheet pile cofferdam
 . permeable stretch in the river: steel piles
- Type GA-10: . impermeable stretch on flood plain: sheet pile cofferdam
 . permeable stretch in the river: steel piles
 . partly submerged at high water stages
- Type GA-11: . same at type GA-9, however, concrete piles instead of steel piles
- Type GA-12: . same as type GA-10, however, concrete piles instead of steel piles

2.3 TEST SITE BAHADURABAD

2.3.1 General

The selected test site in the Bahadurabad area is located about 3.5 km south of Bahadurabad ghat (see Fig.A.20-10). The change of the bankline from May to October 1992 shows that also this area was heavily eroded in 1992 and according to the predictions erosion is further to be expected during the coming years. Since, however, the construction of revetments at this test site is planned for the year 1994 only, bank erosion during 1993 and at the beginning of 1994 will be checked continuously so that adaptation of the test structures to the then prevailing conditions is possible without greater difficulties.

A river bank revetment consists of three components:

- cover layer
- filter layer
- toe protection

The design of these three components depends on the loads, on the behaviour of the subsoil and on the behaviour of the material exposed to current and wave attack. For the **cover layer** the following materials and/or construction methods are in practice

- natural material:
 - grass, turf, reed, willows, trees, timber work, brush wood, sand, clay
- loose units:
 - gravel, rip-rap (stones, boulders), hand-pitched stones, concrete blocks, bricks
- interlocking units:
 - concrete blocks of interlocking geometric shapes, cable connected blocks, blocks connected to geotextiles
- bound systems
 - chemically bonded sand blocks (unburnt bricks)
 - loose units like stones, boulders etc. bound with cement grout or bitumen
- bituminous systems
 - asphalt concrete
 - asphalt mastic
 - open stone asphalt
 - dense stone asphalt
 - lean sand asphalt
- fabric containers
 - tubes, bags, mats filled with sand, gravel, concrete, lean sand asphalt
- wire mesh containers
 - gabions and mattresses filled with stones, boulders, bricks
- concrete slabs
- asphalt slabs

For the **filter layer** the following systems are commonly used:

- granular filters
- lean sand asphalt
- geotextile filters

The following systems of **toe protection** of a sloped revetment are known:

- launching apron consisting of loose material such as boulders, concrete blocks etc. with or without filter layers and of uniform or increasing thickness
- scour prevention mat

Out of the above listed coverlayer systems those being impermeable can be excluded since they are subject to upward directed water pressure during low water season resulting in an increased thickness of this layer. Moreover, some of the systems cannot be constructed under water thus they are also to be ruled out, at least for the protection of bank areas being permanently under water, that means below standard low water level. Consequently the following systems are not further taken into account for bank protection works under water:

- asphalt concrete and asphalt mastic
- open and dense stone asphalt
- lean sand asphalt
- concrete and bitumen slabs

Open stone asphalt and lean sand asphalt can, however, be constructed above the low water level and the ground water level respectively.

The use and effectiveness of natural material is strongly dependent on their location in relation to the river water level and the ground water level as well. Vegetation is frequently used as protection in the horizon above the conventional revetment against wave or current action at extreme water levels. It should, however, be considered that the natural protection may take several growing seasons to reach the design standard of protection provided it is not destroyed in the meantime. This type of cover should mainly be used in the upper part of the bank and on its landside only, and is therefore, not considered as a permanent protection measure and thus not discussed in the following.

Above low water level and ground water level the construction of vertical walls is also possible without any great difficulty from the technical point of view. Use can be made of sheet pile walls which need, however, be anchored or of with boulders or bricks filled wire mesh gabions.

To sum up one can state that the systems below seem feasible:

1. Above low water level/ground water level

(a) Vertical wall

V 1	sheet pile wall (*)
V 2	wire mesh gabions filled with bricks
V 3	wire mesh gabions filled with boulders

(b) Coverlayer

A 1	rip-rap with stones or boulders
A 2	rip-rap with chemically bonded sand blocks
A 3	rip-rap with cement grout
A 4	rip-rap with bitumen grout
A 5	hand-pitched stone (*)
A 6	concrete blocks, regularly placed
A 7	concrete blocks of interlocking geometric shapes (*)
A 8	open stone asphalt
A 9	lean sand asphalt (*)
A 10	wire mesh gabions filled with bricks
A 11	wire mesh gabions filled with boulders

(c) Filter layer

FA 1	granular filter
FA 2	lean sand asphalt
FA 3	geotextile filter (one layer)
FA 4	geotextile filter (multi-layer)

2. Below low water level/ground water level

(a) Coverlayer

B 1	rip-rap with boulders
B 2	rip-rap with chemically bonded sand blocks
B 3	rip-rap with cement grout (*)
B 4	rip-rap with bitumen grout (*)
B 5	concrete blocks
B 6	cable connected blocks
B 7	blocks connected to geotextiles
B 8	open stone asphalt as prefabricated mattress (*)
B 9	geotextile bags filled with sand/lean sand asphalt
B 10	tubular fabrics filled with sand/lean sand asphalt
B 11	tubular fabrics filled with concrete (*)
B 12	wire mesh mattresses filled with stones/boulders

(b) Filter layer

FB 1	geotextile filter (one layer)
FB 2	geotextile filter (multi layer)
FB 3	geotextile - sand mat

(c) Toe protection

T 1	boulders
T 2	concrete blocks
T 3	geotextile bags, partially filled with sand
T 4	cable connected blocks as extension of revetment
T 5	tubular fabrics filled with sand or lean sand asphalt as extension of revetment
T 6	wire mesh mattresses filled with stones/bricks
T 7	scour prevention mat

The material needed for the construction of the above listed systems was found after detailed investigations to be either available in Bangladesh or can be procured without any greater difficulties, except the steel sheet piles for the construction of verticle walls above the low water/ground water level and the geotextile filters.

Since the import of steel sheet piles for a relatively small portion of bank protection, which, however, would have to be anchored by piles or ground anchors seems to be to costly, this solution (V 1) was ruled out.

Geotextile filters are up to this day not manufactured in Bangladesh. However, there is no way out of the necessity of this material for the protection of the under water slope of the revetment against erosion and the migration of soil particles out of the sublayers. Granular filters are not suitable for under water construction, specially at greater water depth. Moreover, it was found out that at least 3 layers of different grain sizes are to be installed. The required sandy and coarse material is neither available in the country nor could the whole filter be built in a proper and controlled manner under the prevailing conditions at the Jamuna river.

Out of the group of cover layers above the low water level/ground water level the systems below were excluded from use for the following reasons:

- the construction of concrete blocks of interlocking geometric shapes (A 7) can of course easily be done, however, controlled placing of them on the slope seems to be rather time consuming just as placing of hand-pitched stone (A 5). With regard to the restricted construction time of only 8 months for the in total about 1000 m long test structures it is seems to be wise to waive out their use.
- though lean sand asphalt (A 9) is known as a reliable protection measure of river banks it is to be ruled out under the prevailing conditions, since it is to be expected that those parts of revetments and berms above the water level are specially subject to utilization by the population and animals with the risk of serious damages of the cover layer.

Thus the systems mentioned above and marked with (*) are given up. The construction of rip-rap fixed or stabilized by cement or bitumen grout (B 3 and B 4) is from the technical point of view quite possible, requires, however, more sophisticated equipment. Taking furthermore into account that local contractors are without experience with regard to this special construction method, it was decided not to make use of it within the test structures. The same holds also for open stone asphalt to be placed under water as prefabricated mattresses (B 8).

Finally it seems to be advisable not to include the use of tubular fabrics to be filled with concrete (B 11), since the permeability of such a system cannot always definitely guaranteed.

To sum up, out of the group of cover layer systems which can be used under water, 4 nos. are excluded. Thus 8 systems above and 8 systems of armour layers below the water level were finally selected as slope protection which will be constructed as test structures in efficient combination with different types of filter layer and toe protection.

2.3.2 Design Criteria

Based on the results of topographical survey, hydrographical survey, hydrometric measurements, physical and mathematical model tests the following design criteria for the test structures at Bahadurabad were fixed:

(i) **Water levels**

- standard low water level : + 13.3 m PWD
- standard high water level : + 18.8 m PWD
- high flood level : + 21.4 m PWD ($\hat{=}$ SLW + 8.1 m)
(100 years return period)
- design water level : SLW + 7.8 m
(25 years return period)

(ii) **Soil conditions**

- flood plain level : + 18.0 to 19.0 m PWD
- based on the results of boring No. BA-3 and BA-4 the following soil properties can be assumed

γ/γ'	=	18/8 kN/m ³
ϕ'	=	27.5°
c'	=	0 kN/m ²
- relative density:
down to about + 13.0 m PWD : loose
down to about ± 0 m PWD : medium dense
below ± 0 m PWD : dense
- groundwater level : + 15.0 m PWD.

(iii) **Hydraulic loads**

- design velocity : $u = 3.5$ m/s
- significant wave height : $H_s = 1.0$ m
- wave period : $T = 3$ s

(iv) **Scour depth:**

- 5 to 11 m below river bed

The greater depth is to be expected at the upstream termination of the revetment, the lower value further downstream.

(v) Earthquake loads:

Earthquake loads and liquefaction of soil is not considered, since the frequency of occurrence is rather low and cannot be expected during the Project period. Hence the test structures would be "overdesigned" which is not according to the design philosophy that requires a lower level of safety to allow a certain damage of the structures.

2.3.3 Alternative Design Solutions

The objective of the test structures at Bahadurabad is to concentrate on the improvement of certain components of the revetments viz

- . the cover layer;
- . the filter layer, and
- . the toe protection (falling apron)

by applying different design criteria and installing different (partly innovative) materials. With regard to the toe protection it is planned to test and to monitor the behaviour of the falling apron at different water depths being 5.0 m and 9.5 m respectively below SLW. Since it is intended to construct the revetments in 1994 behind the then existing bankline and thus protected against wave and current attack during the construction phase, it is possible to build the upper part of the revetments in the dry that means above the groundwater level which was found during the subsoil investigations in 1992 about 4.0 m below the flood plain. Based on detailed stability investigations and taking into account the above mentioned subsoil and ground- water conditions it was found out that the underwater slope can be 1:3 (vertical : horizontal) and 1:2 as the slope above the groundwater level. No variations are intended except a test stretch with a vertical wall above the water level and a berm on the riverside slope which is to be arranged also above the water level.

The layout of the test structure including its upstream and downstream terminations are shown on Fig.A.20-10 and feasible cross-sections in Fig.A.20-11 and Fig.A.20-12.

The distinctive features of the variations cross-sections are as follows:

- | | |
|------------|-------------------------|
| Type BA-1: | . shallow falling apron |
| | . no berm on the slope |
| Type BA-2: | . shallow falling apron |
| | . berm on the slope |
| Type BA-3: | . shallow falling apron |
| | . vertical wall |
| Type BA-4: | . deep falling apron |
| | . no berm on the slope |

Type BA-5: . deep falling apron
 . berm on the slope

Type BA-6: . deep falling apron
 . vertical wall

3 COST COMPARISON OF ALTERNATIVE DESIGN SOLUTIONS

3.1 INTRODUCTION

In order to be able to discuss the pros and cons of the alternative design solutions arrived at in Chapter 2 above, the costs to be expected have been estimated on the basis of the unit rates compiled in Section 9.3 of the Main Report. The idea of these cost estimates was to determine not only the total costs of various solutions, but to get also a cost breakdown of the different components of the structures.

3.2 GROYNES

The groynes are designed to consist of an impermeable 60 m long section to be built on the flood plain (earth dam or sheet pile cofferdam) and an also 60 m long structure (steel piles or concrete piles) or solid (impermeable) stretch to be constructed as conventional earth dam or sheet pile cofferdam in the river.

To facilitate the comparison of costs of different sections estimated in the following tables, the costs in Tk per individual section and as per alternative design are compiled. They are based on net working prices and do not include costs for mobilisation, site installation, contractor's overheads and profit, provision for price calculation and contingencies:

- | | | | |
|-----|---|---|---------------|
| (1) | Earth dam on the flood plain
(see type GA-3, GA-4, GA-5, GA-6) | : | Tk 1,312,500 |
| (2) | Steel sheet pile cofferdam on the
flood plain
(see type GA-7 to GA-12) | : | Tk 3,794,640 |
| (3) | Concrete sheet pile cofferdam
on the flood plain
(see type GA-7 to GA-12) | : | Tk 3,074,640 |
| (4) | Earth dam in the river
(see type GA-1) | : | Tk 39,994,962 |

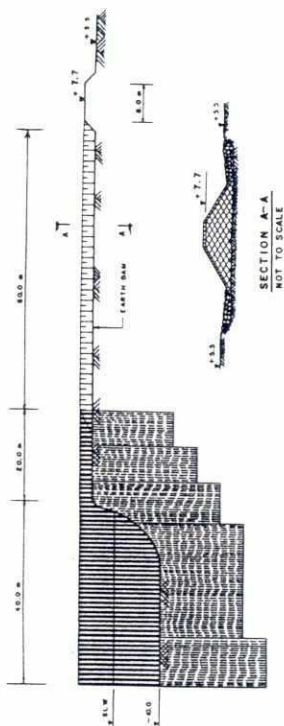
(5)	Steel pile structure (see type GA-3 and GA-9)	:	Tk 33,001,600
(6)	Steel pile structure (see type GA-4 and GA-10)	:	Tk 31,067,200
(7)	Concrete pile structure (see type GA-5 and GA-11)	:	Tk 26,362,000
(8)	Concrete pile structure (see type GA-6 and GA-12)	:	Tk 25,066,000
(9)	Steel sheet pile cofferdam in the river (see type GA-7)	:	Tk 34,040,480
(10)	Steel sheet pile cofferdam in the river (see type GA-8)	:	Tk 30,896,160

3.3 REVETMENTS

In Subsection 2.3.3 six feasible cross sections for the embankments have been selected on which different systems of cover layers, filter layers and toe protection can be installed and tested. Since there is a sheer endless number of possible variations, only one cross section viz type BA-1 has been chosen for cost estimates of the different protection systems leading, however, to 22 alternative solutions, the costs of which are compiled in the following tables showing the total costs and the specific costs (Tk/lm).

Without going into the details it is to be stated that the total cost for the revetment of type BA-1 may be in the range of about 10.1 and 21.0 million Tk depending on the material used for the components of the revetment. They are based on net working prices and do not include costs for mobilisation, site installation, contractor's overheads and profit, provision for price calculation and contingencies.

DESCRIPTION:

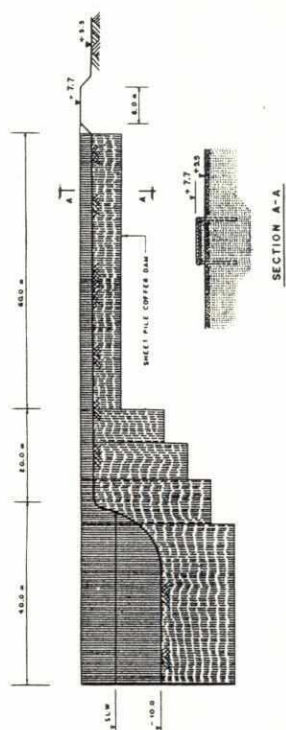


**EARTH DAM
ON THE FLOOD PLAIN**

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES		COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm) Total Tk
1	2	3	4	5	6	7	8	9 10
1	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	32.5	60	1950	1137.5 68,250
2	Compaction with vibration plate	G7	m ³	10	32.5	60	1950	325 19,500
3	Supplying and placing of geosynthetic filter mat above water Type 1	AF5	m ²	150	9	90	810	1350 121,500
4	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	AP4	m ²	925	9	90	810	8325 749,250
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	G11	m ³	2800	4	30	120	11200 336,000
6	Placing of C.C. blocks above water by crane (various sizes)	AP8	m ²	120	5	30	150	600 18,000
							Total	22937.5 1,312,500

**Table A.20.3.2-1 : Cost estimate for earth dam on the flood plain
(type GA-3 to GA-6)**

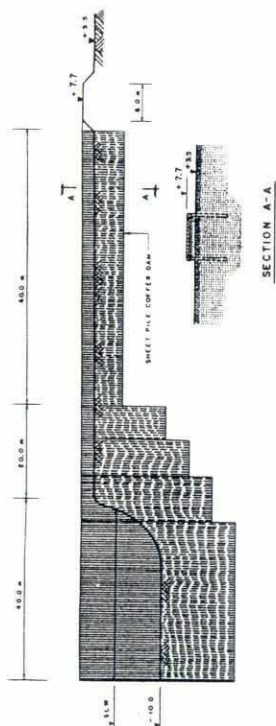
DESCRIPTION:

STEEL SHEET PILE COFFERDAM
ON THE FLOOD PLAIN

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	20	60	1200	700	42,000
2	Compaction with vibration plate	G7	m ³	10	20	60	1200	200	12,000
3	Supply, spreading & levelling of coarse sand t = 0.15m	AF1	m ²	140	8	60	480	1120	67,200
4	Herring bone bond bricks in 2 layer encased with two layer wire mesh	AP1	m ²	403	8	60	480	3224	193,440
5	Supplying & sinking steel sheet piles (82 kg/m ²), t = 8 mm	P6	m ²	4000	6	120	720	24000	2,880,000
6	Bracing of steel or C.C. piles	P9	m	5000	1	120	120	5000	600,000
							Total	34244	3,794,640

Table A.20.3.2-2 : Cost estimate for steel sheet pile cofferdam on the flood plain
(type GA-7 to GA-12)

DESCRIPTION:

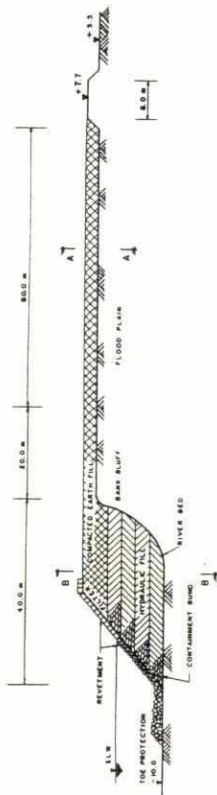


**CONCRETE SHEET PILE COFFERDAM
ON THE FLOOD PLAIN**

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	20	60	1200	700	42,000
2	Compaction with vibration plate	G7	m ³	10	20	60	1200	200	12,000
3	Supply, spreading & levelling of coarse sand t = 0.15m	AF1	m ²	140	8	60	480	1120	67,200
4	Herring bone bond bricks in 2 layer encased with two layer wire mesh	AP1	m ²	403	8	60	480	3224	193,440
5	Supplying & sinking C.C. sheet piles (6x0.75x0.2)	P8	m ²	3000	6	120	720	18000	2,160,000
6	Bracing of steel or C.C. piles	P9	m	5000	1	120	120	5000	600,000
							Total	28244	3,074,640

**Table A.20.3.2-3 : Cost estimate for concrete sheet pile cofferdam on the flood plain
(type GA-7 to GA-12)**

DESCRIPTION:

EARTH DAM
IN THE RIVER

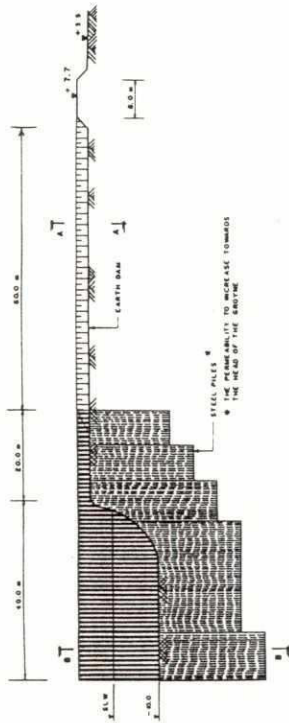
ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Hydraulic excavation by dredger 0 - 3000m 3000m - 4000m 4000m - 5000m	G4	m ²	65	744	40	29760	48360	1,934,400
2	Supply & placing of earth filled gunny bags under water as containment bund used as hydraulic fill dam @ 21 m ³ /m of 3m height	G8	m ²	1419	44	106	4664	62436	6,618,216
3	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	32.5	80	2600	1137.5	91,000
4	Excavation by Dragline/Excavator	G3	m ³	67	144	20	2880	9648	192,960
5	Compaction with vibration plate	G7	m ³	10	32.5	80	2600	325	26,000
6	Spreading, Levelling & Compaction of excavated soil (3 layers)	G6	m ³	51	144	20	2880	7344	146,880
7	Slope preparation above water by excavator	G16	m ²	40	22	73	1606	880	64,240
8	Supplying and placing of geosynthetic filter mat above water Type 1	AF5	m ²	650	6	130	780	3900	507,000
9	Supplying and placing of geosynthetic filter mat above water Type 2	AF4	m ²	135	22	73	1606	2970	216,810

Table A.20.3.2-4 : Cost estimate for earth dam in the river
(type GA-1)

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST		
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk	
1	2	3	4	5	6	7	8	9	10	
10	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	BF2	m ²	346	44	106	4664	15224	1,613,744	
11	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	G11	m ³	2800	14	203	2842	39200	7,957,600	
12	Placing of C.C. blocks above water by crane (various sizes)	AP8	m ²	120	28	203	5684	3360	682,080	
13	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	G11	m ³	2800	22	106	2332	61600	6,529,600	
14	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	BP2	m ²	500	44	106	4664	22000	2,332,000	
15	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	G11	m ³	2800	19.2	197	3782.4	53760	10,590,720	
16	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	19.2	197	3782.4	2496	491,712	
							Total	334640.5	39,994,962	

Table A.20.3.2-4 (continued) : Cost estimate for earth dam in the river
(type GA-1)

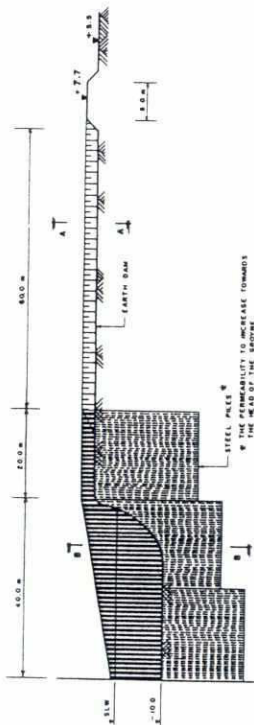
DESCRIPTION:

STEEL PILE WORKS
IN THE RIVER

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking steel sheet piles dia 50mm, t = 10mm	P4	m/m ²	8060	28	120	3360	225680	27,081,600
2	Bracing of steel or C.C. piles	P9	m	5000	5	60	300	25000	1,500,000
3	Supply & placing of geosynthetic sand bags (0.5 m ³) 1.2x1.3 = 1.5 m ² , Type 5	G10	m ³	1570	40	65	2600	62800	4,082,000
4	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	318680	33,001,600

Table A.20.3.2-5 : Cost estimate for steel pile structure
(type GA-3 and GA-9)

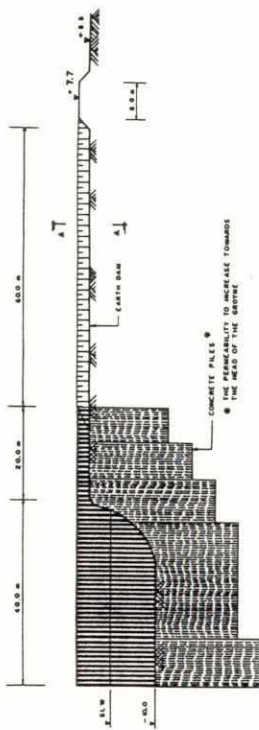
DESCRIPTION:

STEEL PILE WORKS
IN THE RIVER

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre lm	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking steel sheet piles dia 50mm, t = 10mm	P4	m/m ²	8060	26	120	3120	209560	25,147,200
2	Bracing of steel or C.C. piles	P9	m	5000	5	60	300	25000	1,500,000
3	Supply & placing of geosynthetic sand bags (0.5 m ³) 1.2x1.3 = 1.5 m ² , Type 5	G10	m ³	1570	40	65	2600	62800	4,082,000
4	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	302560	31,067,200

Table A.20.3.2-6 : Cost estimate for steel pile structure
(type GA-4 and GA-10)

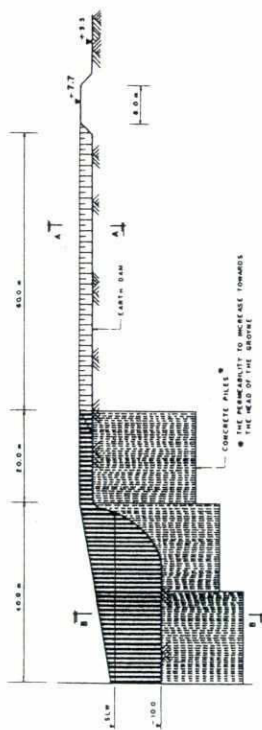
DESCRIPTION:

CONCRETE PILE WORKS
IN THE RIVER

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking of R.C.C. piles dia 50cm,	P5	m ²	5400	28	120	3360	151200	18,144,000
2	Bracing of steel or C.C. piles	P9	m	5000	2	60	120	10000	600,000
3	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	G11	m ³	2800	40	65	2600	112000	7,280,000
4	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	278400	26,362,000

Table A.20.3.2-7 : Cost estimate for concrete pile structure
(type GA-5 and GA-11)

DESCRIPTION:

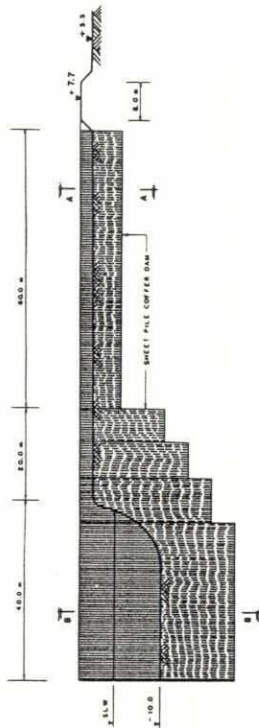


CONCRETE PILE WORKS IN THE RIVER

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking of R.C.C. piles dia 50cm	P5	m ²	5400	26	120	3120	140400	16,848,000
2	Bracing of steel or C.C. piles	P9	m	5000	2	60	120	10000	600,000
3	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	G11	m ³	2800	40	65	2600	112000	7,280,000
4	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	267600	25,066,000

Table A.20.3.2-8 : Cost estimate for concrete pile structure
(type GA-6 and GA 12)

DESCRIPTION:

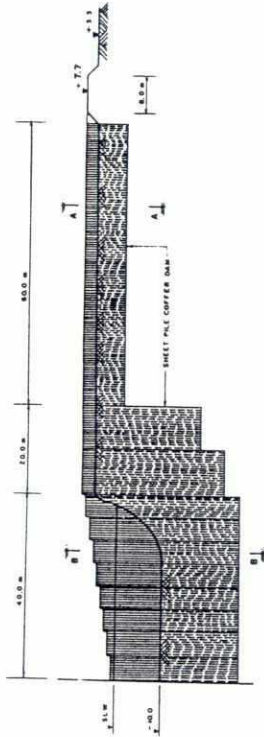


IMPARMEABLE GROUYNE
STEEL SHEET PILE COFFERDAM
IN THE RIVER

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking of steel sheet piles (155 kg/m ²), t = 12.8 mm	P7	m ²	7595	28	128	3584	212660	27,220,480
2	Bracing of steel or C.C. piles	P9	m	5000	4	120	480	20000	2,400,000
3	Supply & placing of geosynthetic sand bags (0.5 m ³) 1.2x1.3 = 1.5 m ² , Type 5	G10	m ³	1570	40	65	2600	62800	4,082,000
4	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	300660	34,040,480

Table A.20.3.2-9 : Cost estimate for steel sheet pile cofferdam structure
(type GA-7)

DESCRIPTION:



**IMPARMEABLE GROUYNE
STEEL SHEET PILE WORKS
IN THE RIVER**

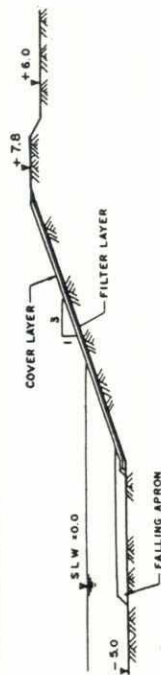
ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking of steel sheet piles (155 kg/m ²), t = 12.8 mm	P7	m ²	7595	26	128	3328	197470	25,276,160
2	Bracing of steel or C.C. piles	P9	m	5000	2	120	240	10000	1,200,000
3	Supply & placing of geosynthetic sand bags (0.5 m ³) 1.2x1.3=1.5 m ² , Type 5	G10	m ³	1570	40	65	2600	62800	4,082,000
4	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	275470	30,896,160

**Table A.20.3.2-10 : Cost estimate for steel sheet pile cofferdam structure
(type GA-8)**

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T1

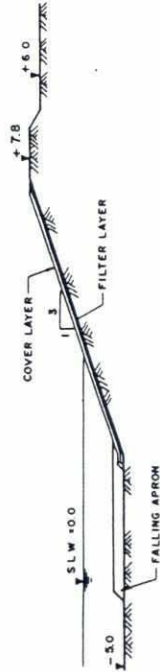
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	21	100	2100	6300	630,000
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	T1	BP1	m ³	1870	45	100	4500	84150	8,415,000
								Total	130790	13,079,000

Table A.20.3.3-1 : Cost estimate for embankment type BA-1, alternative No. 1

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



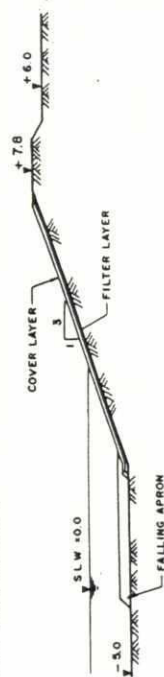
FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T1

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	T1	BP1	m ³	1870	45	100	4500	84150	8,415,000
								Total	131756	13,175,600

Table A.20.3.3-2 : Cost estimate of embankment type BA-1, alternative No. 2

DESCRIPTION:

- TYPE BA-1
- SHALLOW FALLING APRON
 - NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB3
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T1

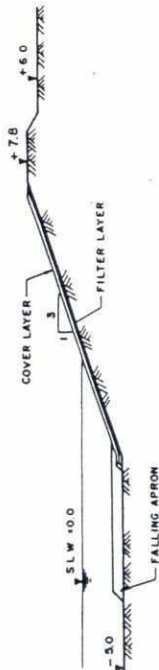
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	21	100	2100	10395	1,039,500
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	T1	BP1	m ³	1870	45	100	4500	84150	8,415,000
								Total	134885	13,488,500

Table A.20.3.3-3 : Cost estimate of embankment type BA-1, alternative No. 3

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB1
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T3

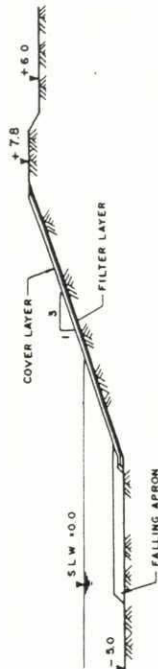
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	21	100	2100	6300	630,000
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supply & dumping of geotextile sand bags (G10) + (BP4)	T1	BP7	m ³	1700	45	100	4500	76500	7,650,000
								Total	123140	12,314,000

Table A.20.3.3-4 : Cost estimate of embankment type BA-1, alternative No. 4

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3

FILTER B W : FB2

COVER A W : A1

COVER B W : B1

TOE PROTECTION : T3

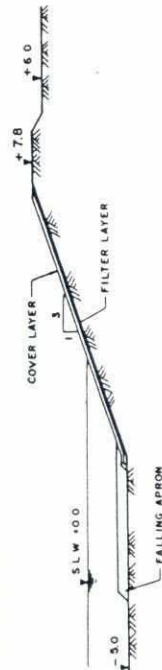
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supply & dumping of geotextile sand bags (G10) + (BP4)	T1	BP7	m ³	1700	45	100	4500	76500	7,650,000
								Total	124106	12,410,600

Table A.20.3.3-5 : Cost estimate of embankment type BA-1, alternative No. 5

DESCRIPTION:

TYPE BA - I

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB3
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T3

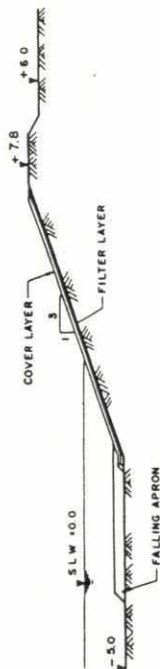
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	21	100	2100	10395	1,039,500
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supply & dumping of geotextile sand bags (G10) + (BP4)	T1	BP7	m ³	1700	45	100	4500	76500	7,650,000
								Total	127235	12,723,500

Table A.20.3.3-6 : Cost estimate of embankment type BA-1, alternative No. 6

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB1
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T7

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	21	100	2100	6300	630,000
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supply and placing of geotextile scour prevention mat Type 4	T7	BP3	m ³	1300	45	100	4500	58500	5,850,000
								Total	105140	10,514,000

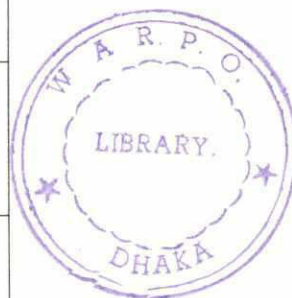
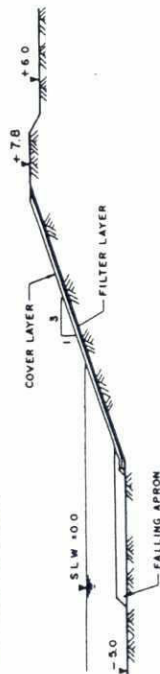


Table A.20.3.3-7 : Cost estimate of embankment type BA-1, alternative No. 7

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T7

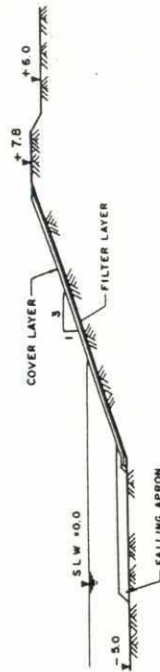
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ²	935	19	100	1900	17765	1,776,500
5	Supply and placing of geotextile scour prevention mat Type 4	T7	BP3	m ²	1300	45	100	4500	58500	5,850,000
								Total	106106	10,610,600

Table A.20.3.3-8 : Cost estimate of embankment type BA-1, alternative No. 8

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB3
 COVER A W : A1
 COVER B W : B1
 TOE PROTECTION : T7

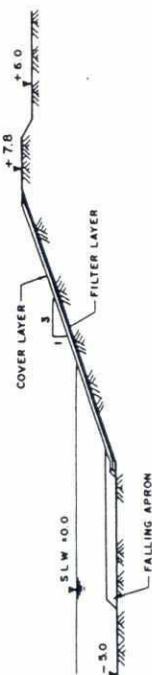
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	UNIT	PRICE NO	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	5	4	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	m ²	AF5	150	21	100	2100	3150	315,000
2	Supply & placing of geosynthetic sand mat Type 3	FB3	m ²	BF3	495	21	100	2100	10395	1,039,500
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	m ²	AP4	925	21	100	2100	19425	1,942,500
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	m ²	BP1	935	19	100	1900	17765	1,776,500
5	Supply and placing of geotextile scour prevention mat Type 4	T7	m ²	BP3	1300	45	100	4500	58500	5,850,000
								Total	109235	10,923,500

Table A.20.3.3-9 : Cost estimate of embankment type BA-1, alternative No. 9

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB1
 COVER A W : A2
 COVER B W : B2
 TOE PROTECTION : T1

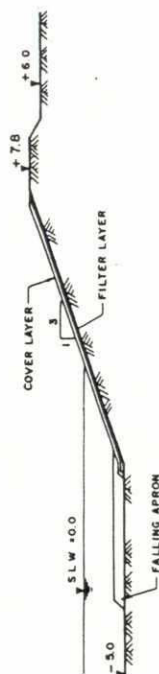
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	21	100	2100	6300	630,000
3	Production of chemical bonded blocks	A2	G14	m ³	1665	10.5	100	1050	17482.5	1,748,250
4	Placing of C.C. blocks above water by crane (various sizes)	A2	AP8	m ²	120	21	100	2100	2520	252,000
5	Production of chemical bonded blocks	B2	G14	m ³	1665	9.5	100	950	15817.5	1,581,750
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B2	BP2	m ²	500	19	100	1900	9500	950,000
7	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	T1	BP1	m ³	1870	45	100	4500	84150	8,415,000
								Total	138920	13,892,000

Table A.20.3.3-10 : Cost estimate of embankment type BA-1, alternative No. 10

DESCRIPTION:

TYPE BA - I

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : A2
 COVER B W : B2
 TOE PROTECTION : T3

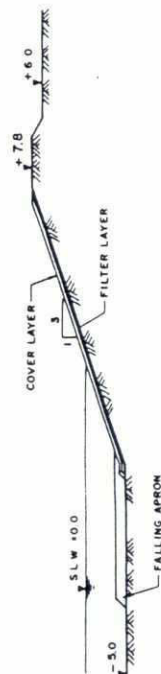
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Production of chemical bonded blocks	A2	G14	m ³	1665	10.5	100	1050	17482.5	1,748,250
4	Placing of C.C. blocks above water by crane (various sizes)	A2	AP8	m ²	120	21	100	2100	2520	252,000
5	Production of chemical bonded blocks	B2	G14	m ³	1665	9.5	100	950	15817.5	1,581,750
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B2	BP2	m ²	500	19	100	1900	9500	950,000
7	Supply & dumping of geotextile sand bags (G10) + (BP4)	T3	BP7	m ³	1700	45	100	4500	76500	7,650,000
								Total	132236	13,223,600

Table A.20.3.3-11 : Cost estimate of embankment type BA-1, alternative No. 11

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB3
 COVER A W : A2
 COVER B W : B2
 TOE PROTECTION : T6

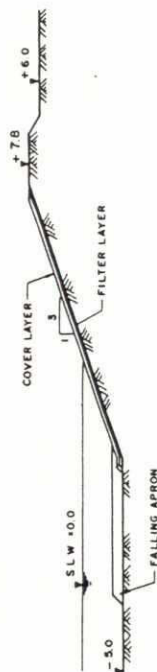
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	21	100	2100	10395	1,039,500
3	Production of chemical bonded blocks	A2	G14	m ³	1665	10.5	100	1050	17482.5	1,748,250
4	Placing of C.C. blocks above water by crane (various sizes)	A2	AP8	m ²	120	21	100	2100	2520	252,000
5	Production of chemical bonded blocks	B2	G14	m ³	1665	9.5	100	950	15817.5	1,581,750
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B2	BP2	m ²	500	19	100	1900	9500	950,000
7	Manufacturing of gabions with boulders (various sizes)	T6	AP11	m ³	2392	45	100	4500	107640	10,764,000
8	Dumping of boulders C.C. blocks, gabions etc. from barges	T6	BP4	m ³	130	45	100	4500	5850	585,000
								Total	172355	17,235,500

Table A.20.3.3-12 : Cost estimate of embankment type BA-1, alternative No. 12

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB1
 COVER A W : A6
 COVER B W : B5
 TOE PROTECTION : T2

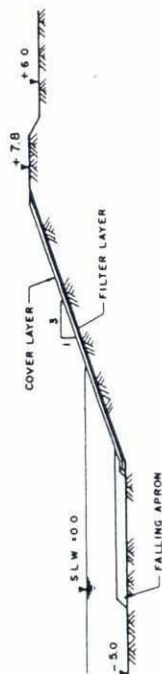
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST		
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total	Tk
1	2	3	4	5	6	7	8	9	10	11	
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000	
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	21	100	2100	6300	630,000	
3	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	m ³	2800	10.5	100	1050	29400	2,940,000	
4	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	21	100	2100	2520	252,000	
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	B5	G11	m ³	2800	9.5	100	950	26600	2,660,000	
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B5	BP2	m ²	500	19	100	1900	9500	950,000	
7	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	T2	G11	m ³	2800	45	100	4500	126000	12,600,000	
8	Dumping of boulders C.C. blocks, gabions etc. from barges	T6	BP4	m ³	130	45	100	4500	5850	585,000	
								Total	209320	20,932,000	

Table A.20.3.3-13 : Cost estimate of embankment type BA-1, alternative No. 13

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : A6
 COVER B W : B5
 TOE PROTECTION : T7

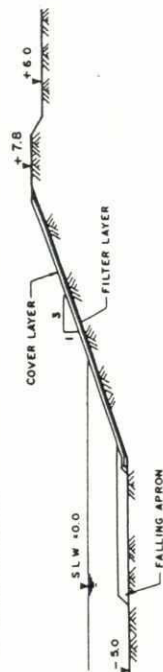
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	m ³	2800	10.5	100	1050	29400	2,940,000
4	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	21	100	2100	2520	252,000
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	B5	G11	m ³	2800	9.5	100	950	26600	2,660,000
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B5	BP2	m ²	500	19	100	1900	9500	950,000
7	Supply & dumping of geotextile sand bags (G10) + (BP4)	T3	BP7	m ³	1700	45	100	4500	76500	7,650,000
								Total	154936	15,493,600

Table A.20.3.3-14 : Cost estimate of embankment type BA-1, alternative No. 14

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB3
 COVER A W : A6
 COVER B W : B5
 TOE PROTECTION : T7

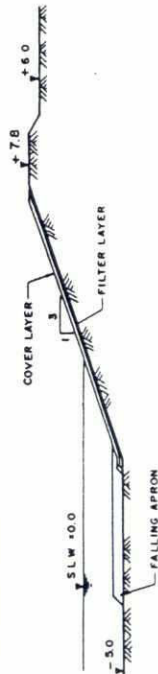
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	21	100	2100	10395	1,039,500
3	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	m ³	2800	10.5	100	1050	29400	2,940,000
4	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	21	100	2100	2520	252,000
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	B5	G11	m ³	2800	9.5	100	950	26600	2,660,000
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B5	BP2	m ²	500	19	100	1900	9500	950,000
7	Supply and placing of geotextile scour prevention mat Type 4	T3	BP3	m ²	1300	15	100	1500	19500	1,950,000
								Total	101065	10,106,500

Table A.20.3.3-15 : Cost estimate of embankment type BA-1, alternative No. 15

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB1
 COVER A W : B6
 COVER B W : B6
 TOE PROTECTION : T2

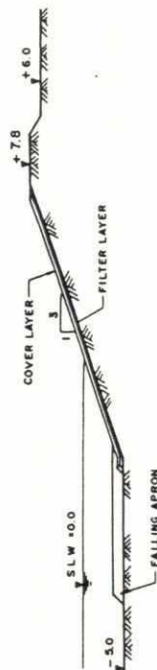
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	21	100	2100	6300	630,000
3	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	B6	BP5	m ²	1475	21	100	2100	30975	3,097,500
4	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	B6	BP5	m ²	1475	19	100	1900	28025	2,802,500
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	T2	G11	m ³	2800	45	100	4500	126000	12,600,000
6	Dumping of boulders C.C. blocks, gabions etc. from barges	T2	BP4	m ³	130	45	100	4500	5850	585,000
								Total	200300	20,030,000

Table A.20.3.3-16 : Cost estimate of embankment type BA-1, alternative No. 16

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : B6
 COVER B W : B6
 TOE PROTECTION : T3

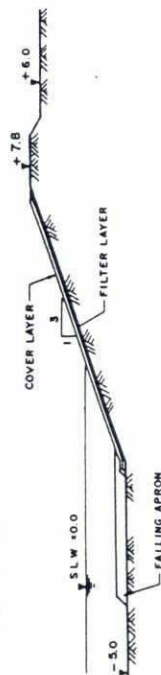
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	B6	BP5	m ²	1475	21	100	2100	30975	3,097,500
4	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	B6	BP5	m ²	1475	19	100	1900	28025	2,802,500
5	Supply & dumping of geotextile sand bags (G10) + (BP4)	T3	BP7	m ³	1700	45	100	4500	76500	7,650,000
								Total	145916	14,591,600

Table A.20.3.3-17 : Cost estimate of embankment type BA-1, alternative No. 17

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W : FB3
 COVER A W : B6
 COVER B W : B6
 TOE PROTECTION : T4

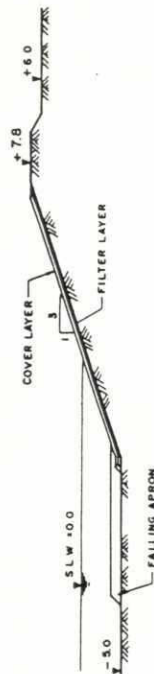
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	21	100	2100	10395	1,039,500
3	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	B6	BP5	m ²	1475	21	100	2100	30975	3,097,500
4	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	B6	BP5	m ²	1475	19	100	1900	28025	2,802,500
5	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	T4	BP5	m ²	1475	45	100	4500	66375	6,637,500
								Total	138920	13,892,000

Table A.20.3.3-18 : Cost estimate of embankment type BA-1, alternative No. 18

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W :
 COVER A W : A6
 COVER B W : B7
 TOE PROTECTION : T2

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	m ³	2800	10.5	100	1050	29400	2,940,000
3	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	21	100	2100	2520	252,000
4	Supply, manufacture and placing of C.C. blocks connected to geotextile t = 0.35m Type 2	B7	BP6	m ²	1774	21	100	2100	37254	3,725,400
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	T2	G11	m ³	2800	45	100	4500	126000	12,600,000
6	Dumping of boulders C.C. blocks, gabions etc. from barges	T2	BP4	m ³	130	45	100	4500	5850	585,000
								Total	204174	20,417,400

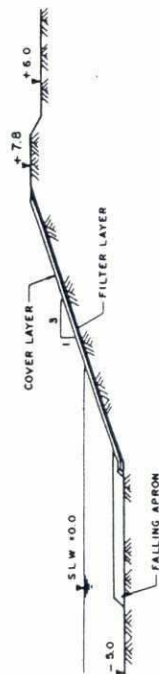
Table A.20.3.3-19 : Cost estimate of embankment type BA-1, alternative No. 19

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DESCRIPTION:

TYPE BA - I

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W :
 COVER A W : A6
 COVER B W : B7
 TOE PROTECTION : T3

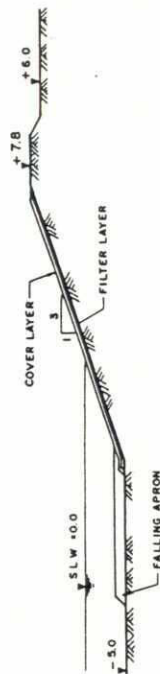
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	sq.m	150	21	100	2100	3150	315,000
2	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	cu.m	2800	10.5	100	1050	29400	2,940,000
3	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	sq.m	120	21	100	2100	2520	252,000
4	Supply, manufacture and placing of C.C. blocks connected to geotextile t = 0.35m Type 2	B7	BP6	sq.m	1774	21	100	2100	37254	3,725,400
5	Supply & dumping of geotextile sand bags (G10) + (BP4)	T3	BP7	cu.m	1700	45	100	4500	76500	7,650,000
								Total	148824	14,882,400

Table A.20.3.3-20 : Cost estimate of embankment type BA-1, alternative No. 20

DESCRIPTION:

TYPE BA-1

- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA3
 FILTER B W :
 COVER A W : A6
 COVER B W : B7
 TOE PROTECTION : T7

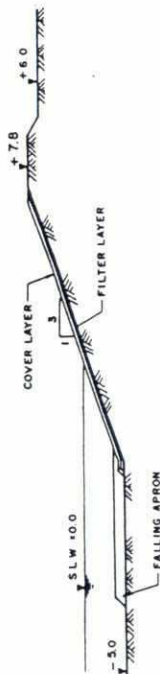
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	m ³	2800	10.5	100	1050	29400	2,940,000
3	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	21	100	2100	2520	252,000
4	Supply, manufacture and placing of C.C. blocks connected to geotextile t = 0.35m Type 2	B7	BP6	m ²	1774	21	100	2100	37254	3,725,400
5	Supply and placing of geotextile scour prevention mat Type 4	T7	BP3	m ²	1300	15	100	1500	19500	1,950,000
								Total	91824	9,182,400

Table A.20.3.3-21 : Cost estimate of embankment type BA-1, alternative No. 21

DESCRIPTION:

TYPE BA-1

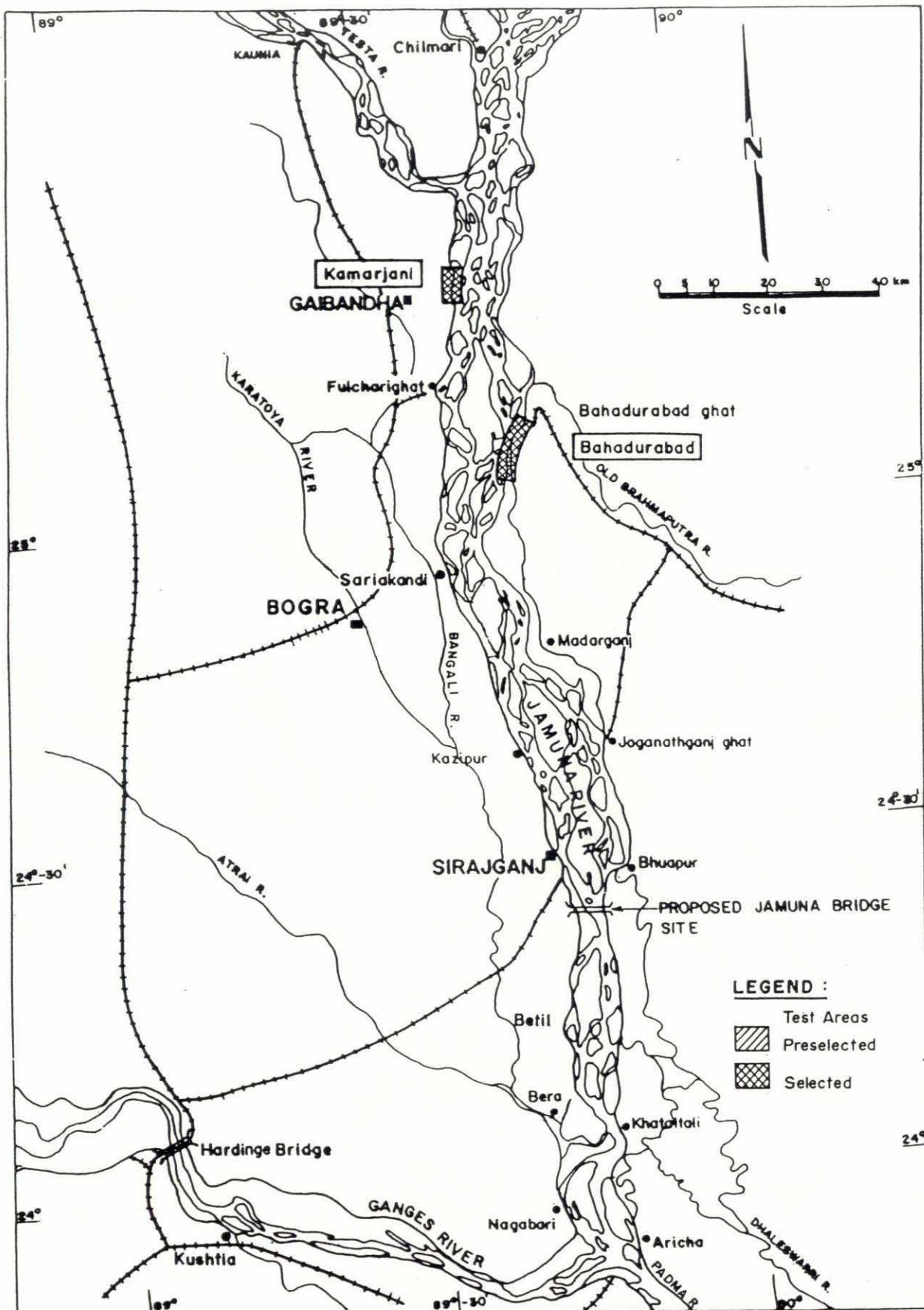
- SHALLOW FALLING APRON
- NO BERM ON SLOPE



FILTER A W : FA2
 FILTER B W : FB2
 COVER A W : A6
 COVER B W : B5
 TOE PROTECTION : T2

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	21	100	2100	7266	726,600
3	Production of C.C. blocks (1:3:6) with brick & coarse sand	A6	G11	m ³	2800	10.5	100	1050	29400	2,940,000
4	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	21	100	2100	2520	252,000
5	Production of C.C. blocks (1:3:6) with brick & coarse sand	B5	G11	m ³	2800	9.5	100	950	26600	2,660,000
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B5	BP2	m ²	500	19	100	1900	9500	950,000
7	Production of C.C. blocks (1:3:6) with brick & coarse sand	T2	G11	m ³	2800	45	100	4500	126000	12,600,000
8	Dumping of boulders C.C. blocks, gabions etc. from barges	T2	BP4	m ³	130	45	100	4500	5850	585,000
								Total	210286	21,028,600

Table A.20.3.3-22 : Cost estimate of embankment type BA-1, alternative No. 22



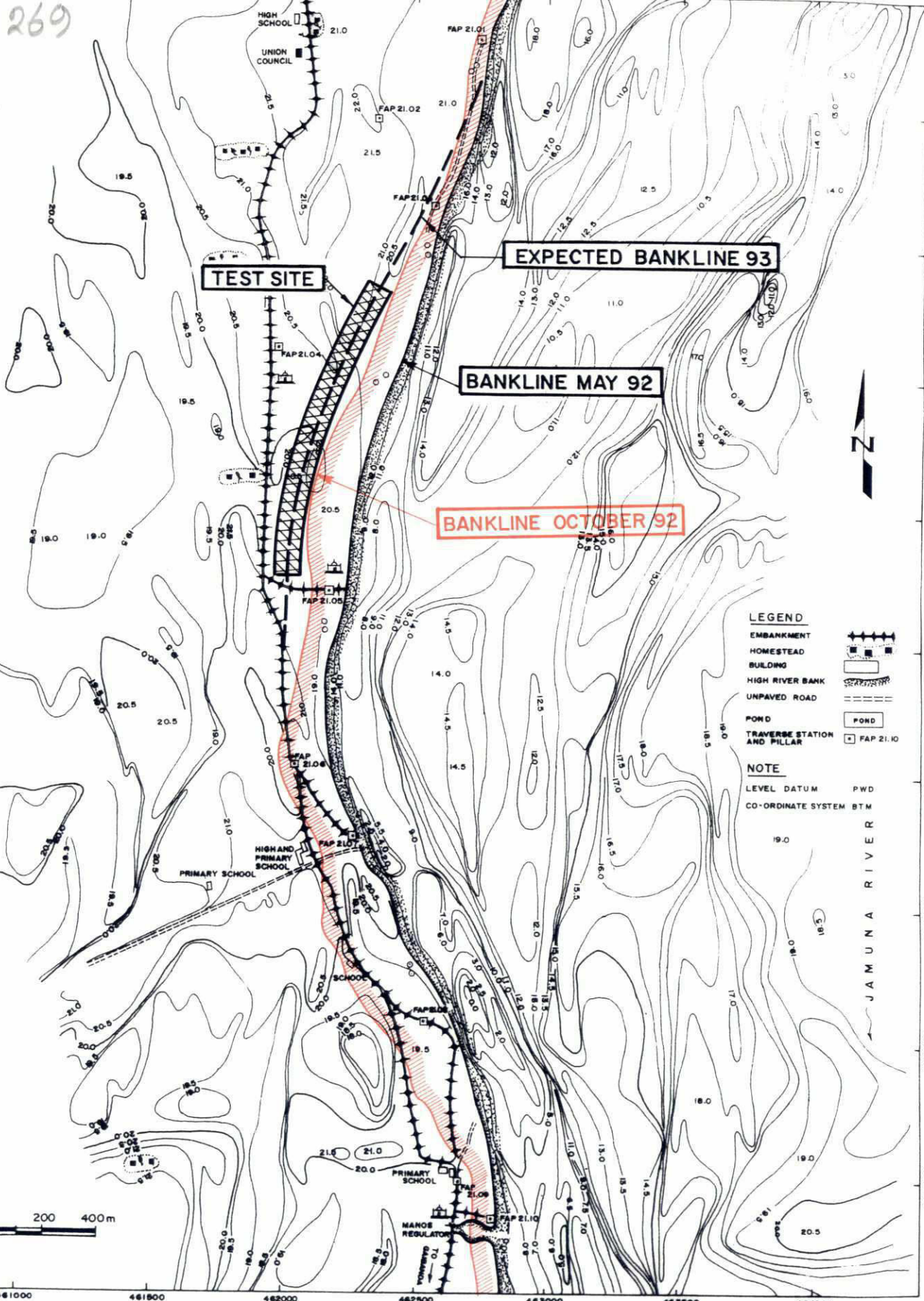
TEST AREAS

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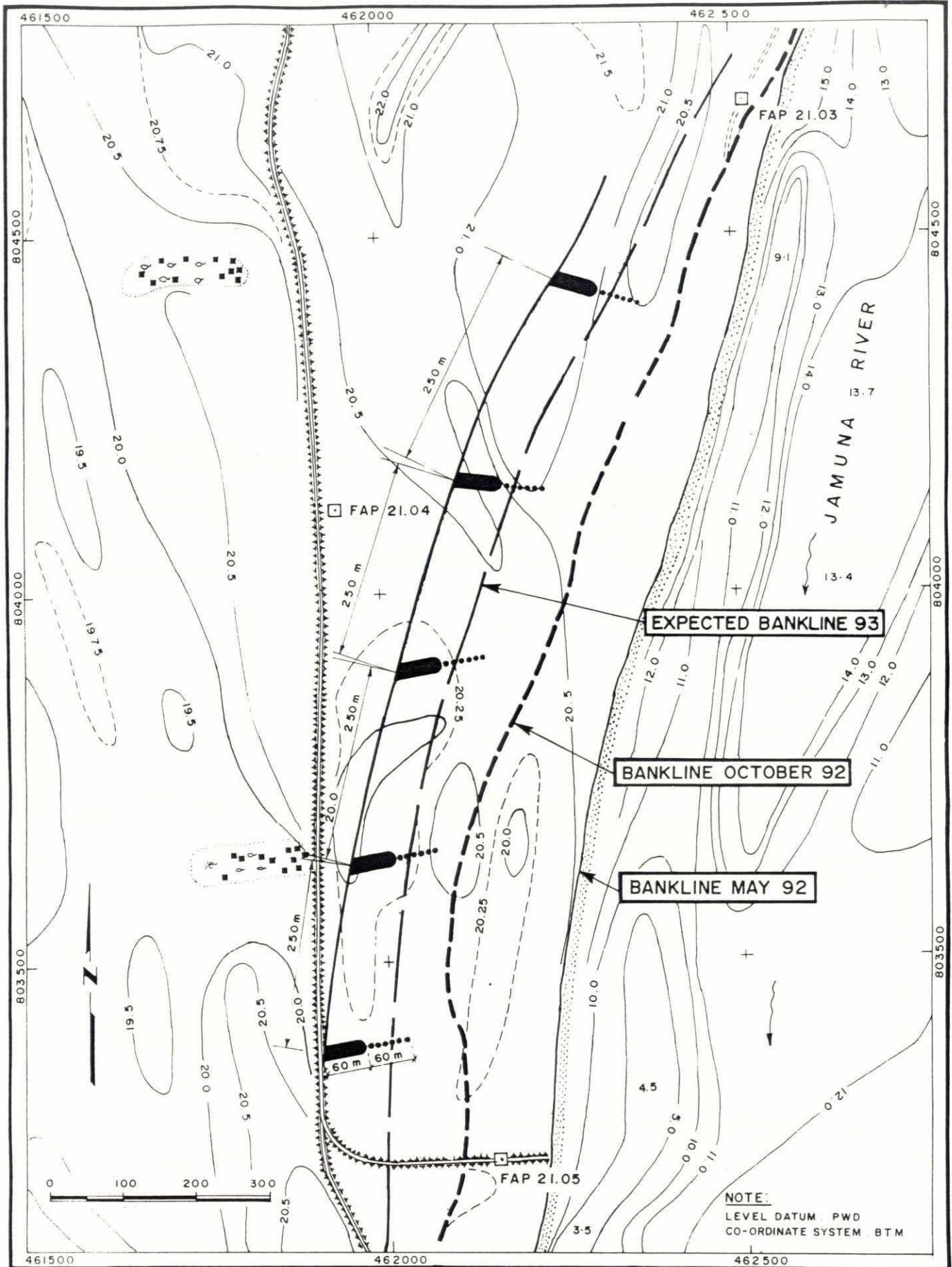
FIG. A.20-1



LOCATION OF TEST SITE
AT KAMARJANI

**BANK PROTECTION AND RIVER TRAINING /
(A FPM) PILOT PROJECT**

FIG. A.20-2



LAYOUT OF GROYNES
AT KAMARJANI TEST SITE
GENERAL LAYOUT

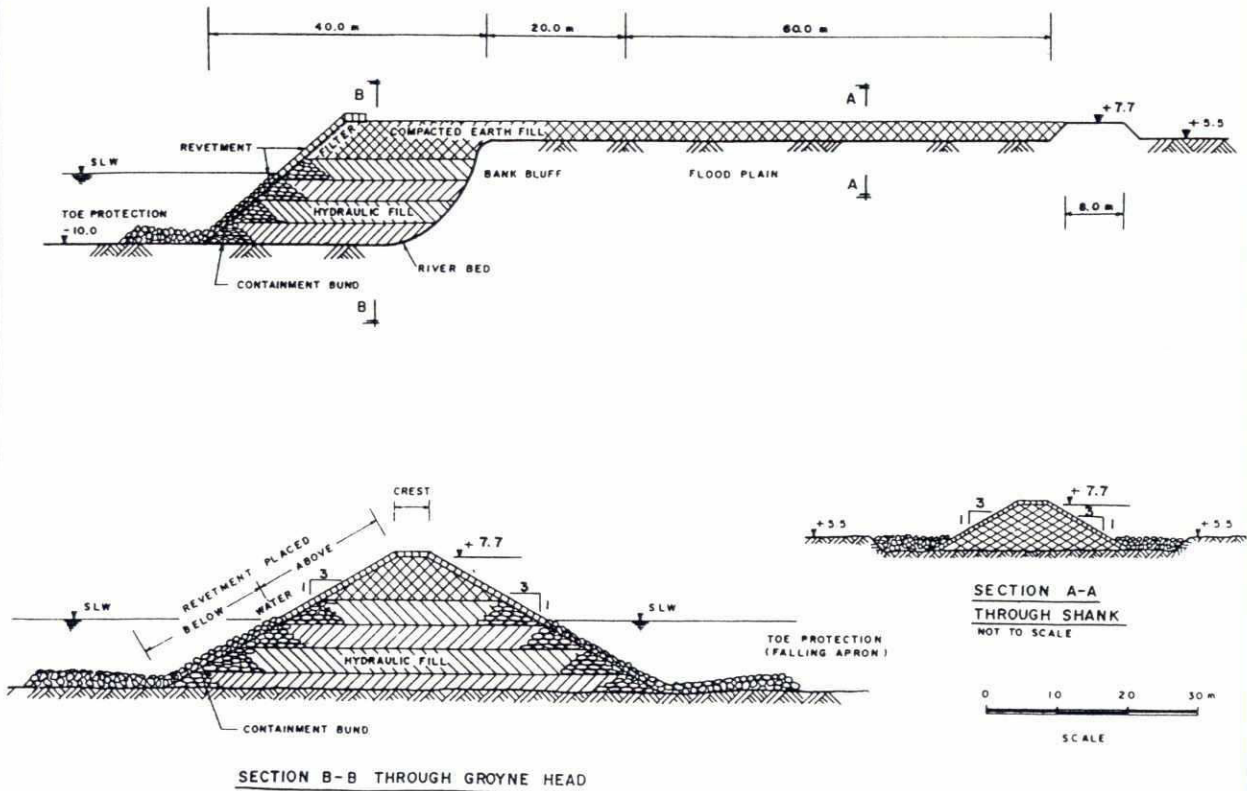
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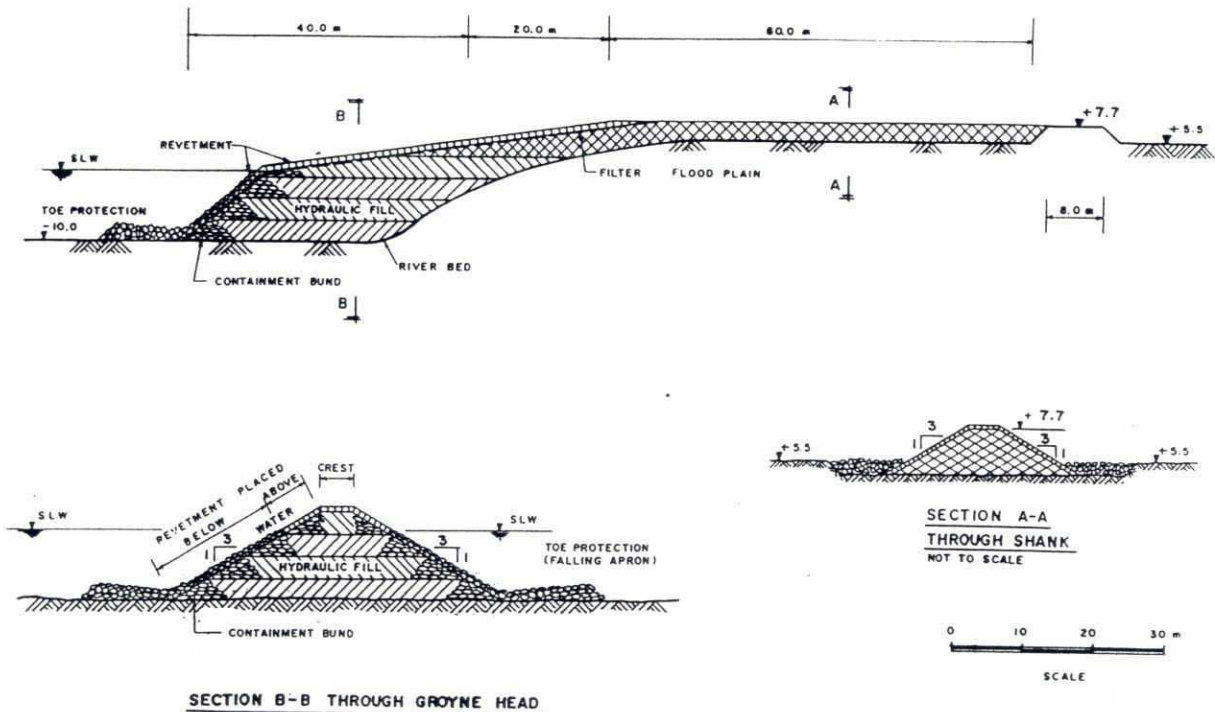
FIG. A. 20-3

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TYPE GA-1



TYPE GA-2



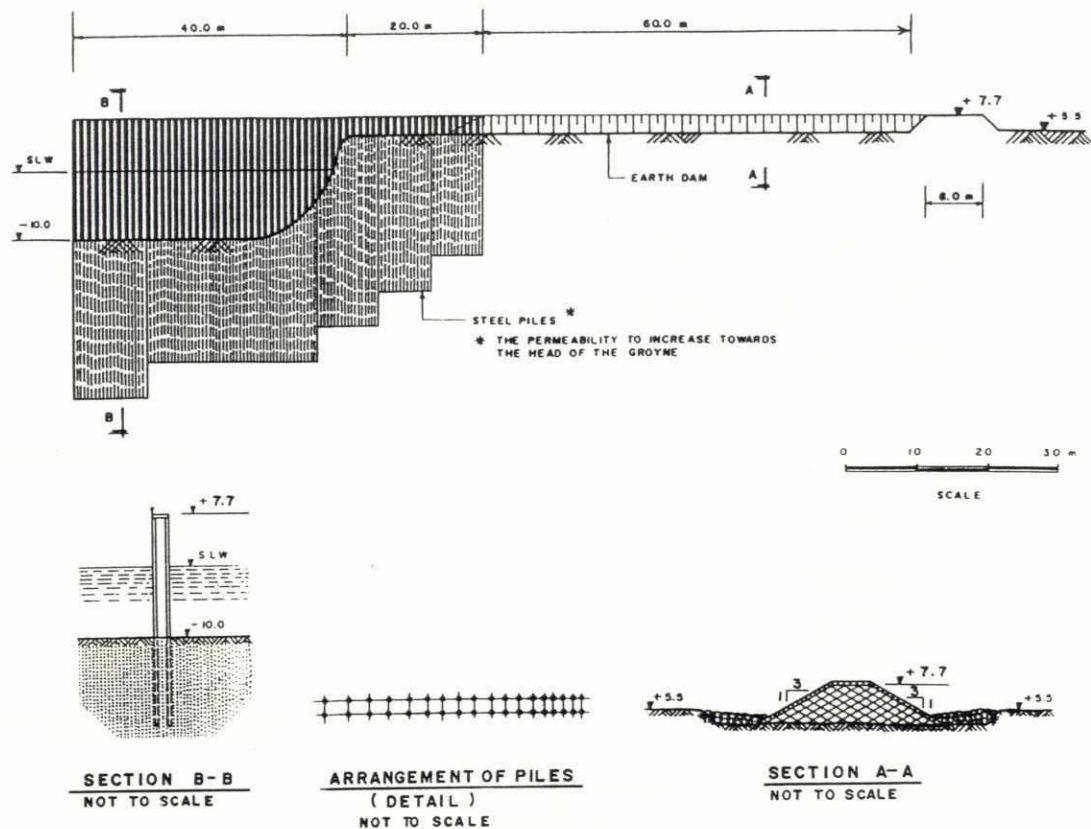
ALTERNATIVE GROYNES
AT KAMARJANI
TEST SITE

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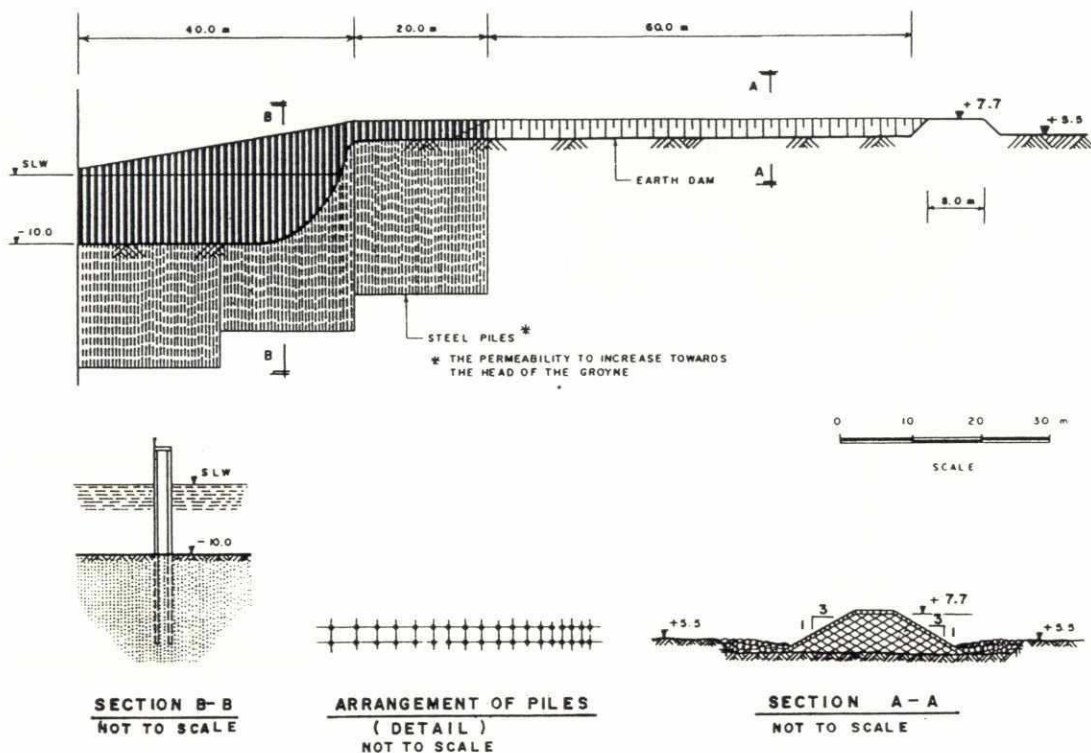
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FIG. A.20 - 4

TYPE GA-3



TYPE GA-4



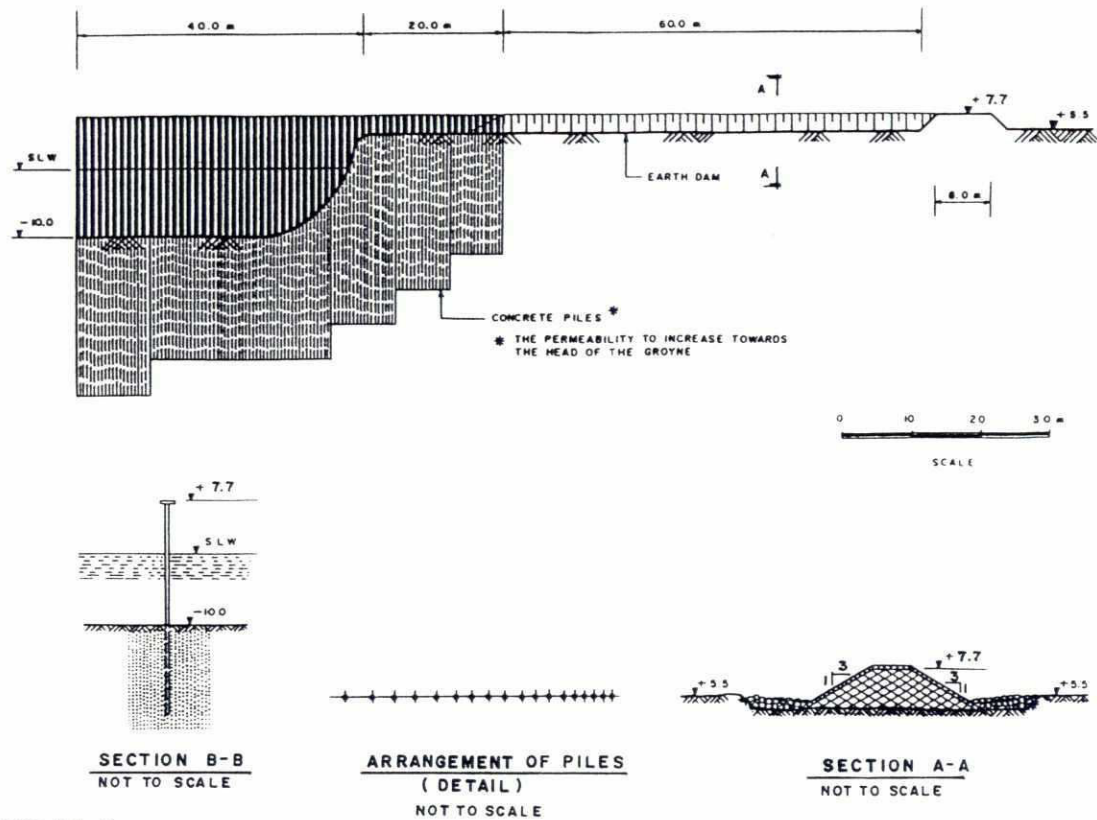
ALTERNATIVE GROYNES
AT KAMARJANI
TEST SITE

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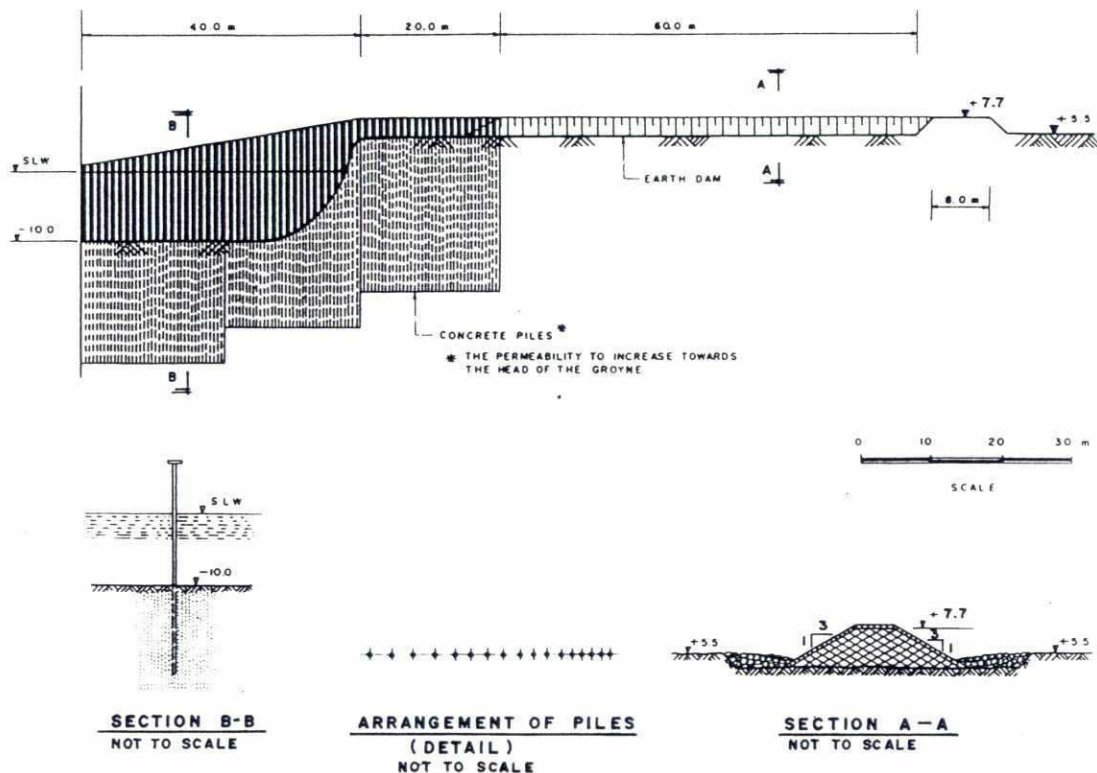
FINAL REPORT

FIG. A.20-5

TYPE GA-5



TYPE GA-6



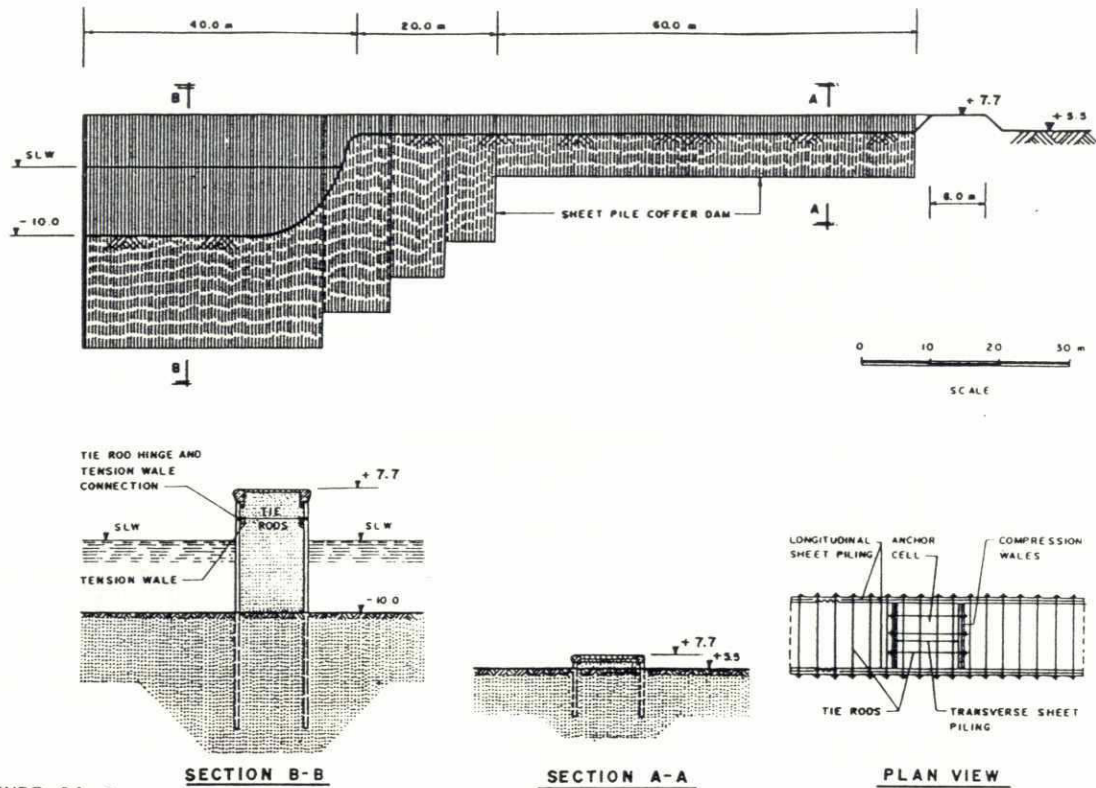
ALTERNATIVE GROYNES
AT KAMARJANI
TEST SITE

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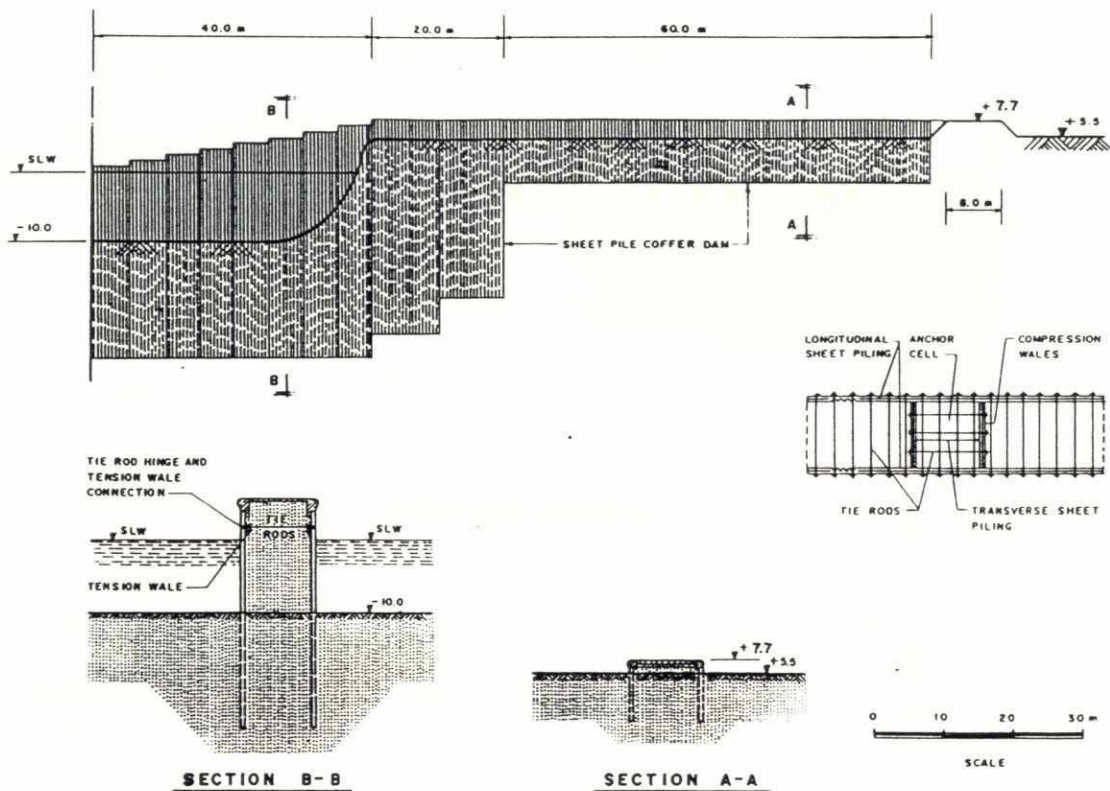
FINAL REPORT

FIG. A.20-6

TYPE GA-7



TYPE GA-8



ALTERNATIVE GROYNES
AT KAMARJANI
TEST SITE

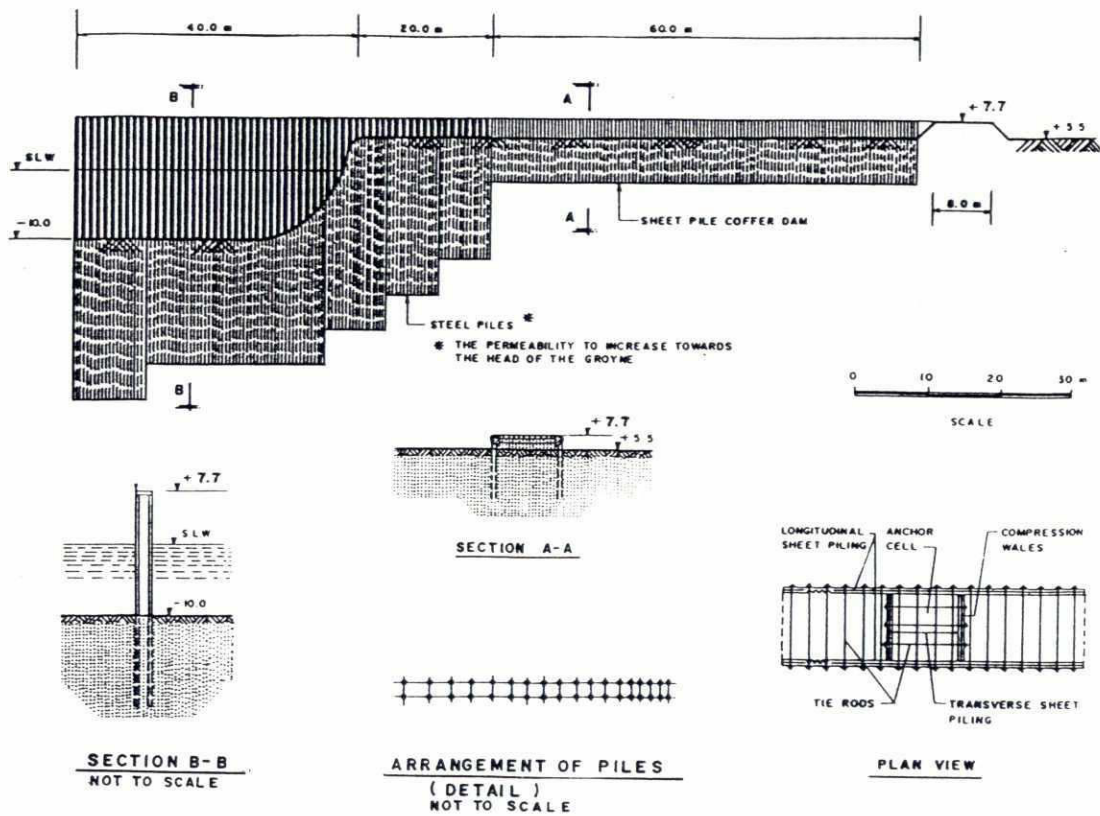
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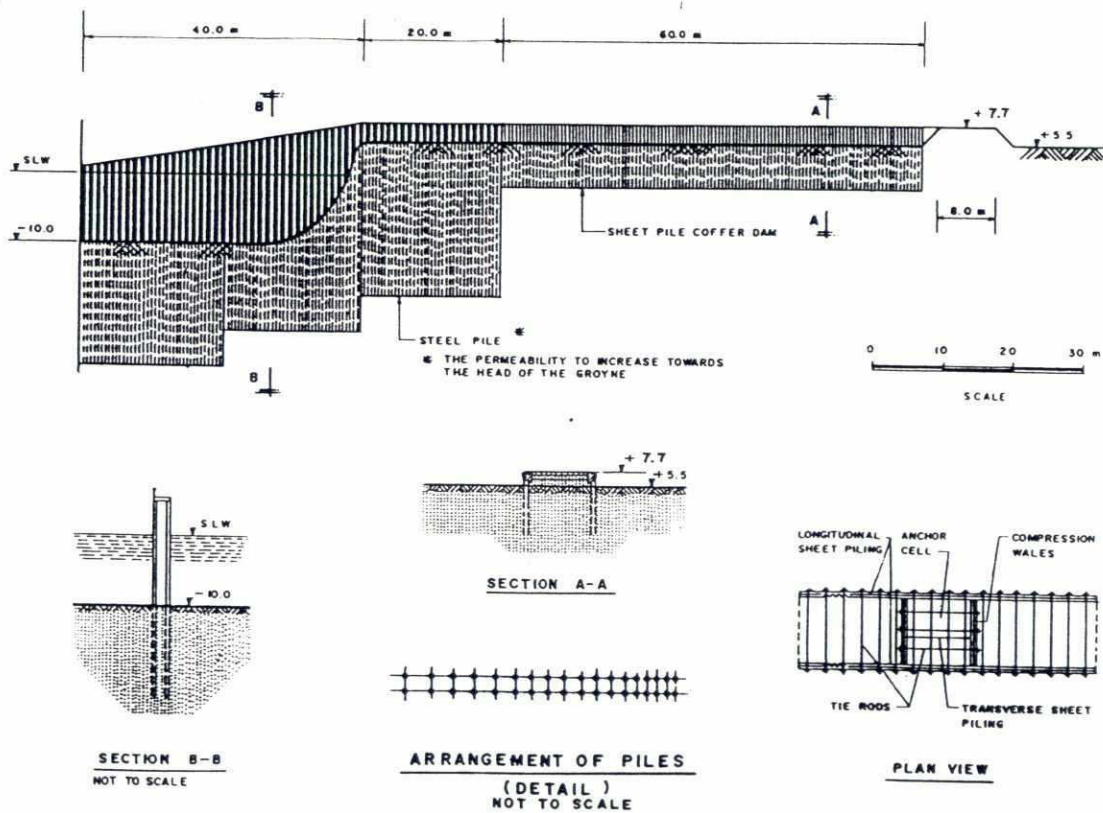
FIG. A.20-7

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TYPE GA-9



TYPE GA-10



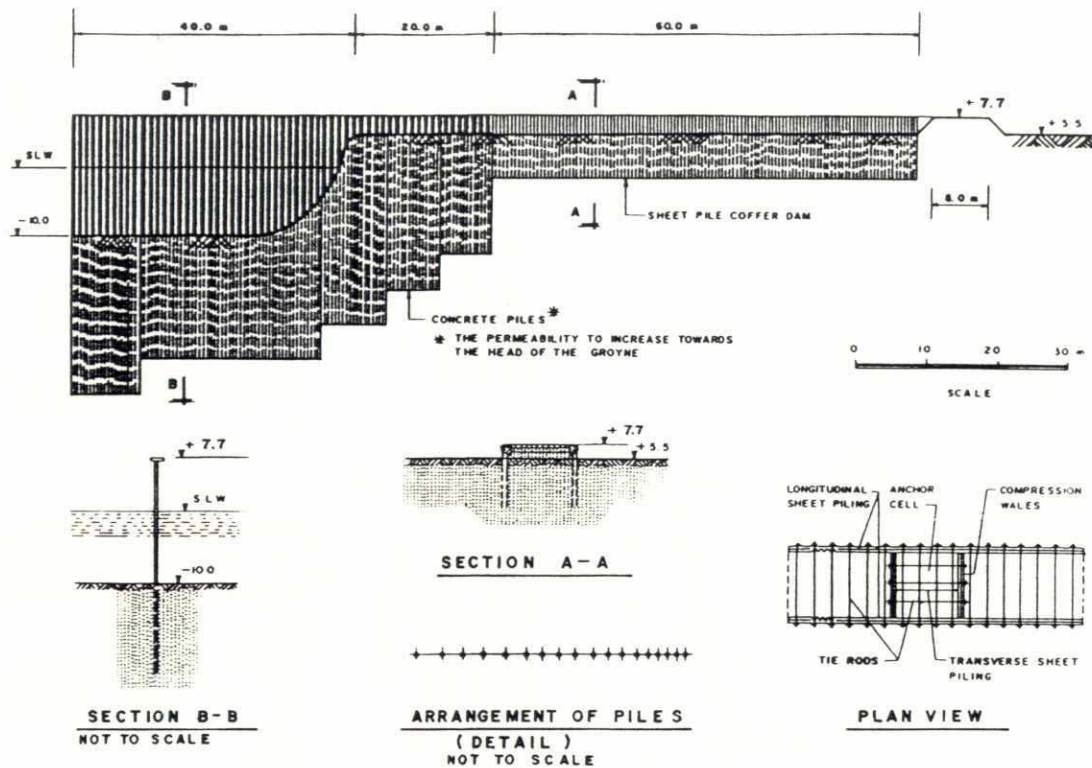
ALTERNATIVE GROYNES
AT KAMARJANI
TEST SITE

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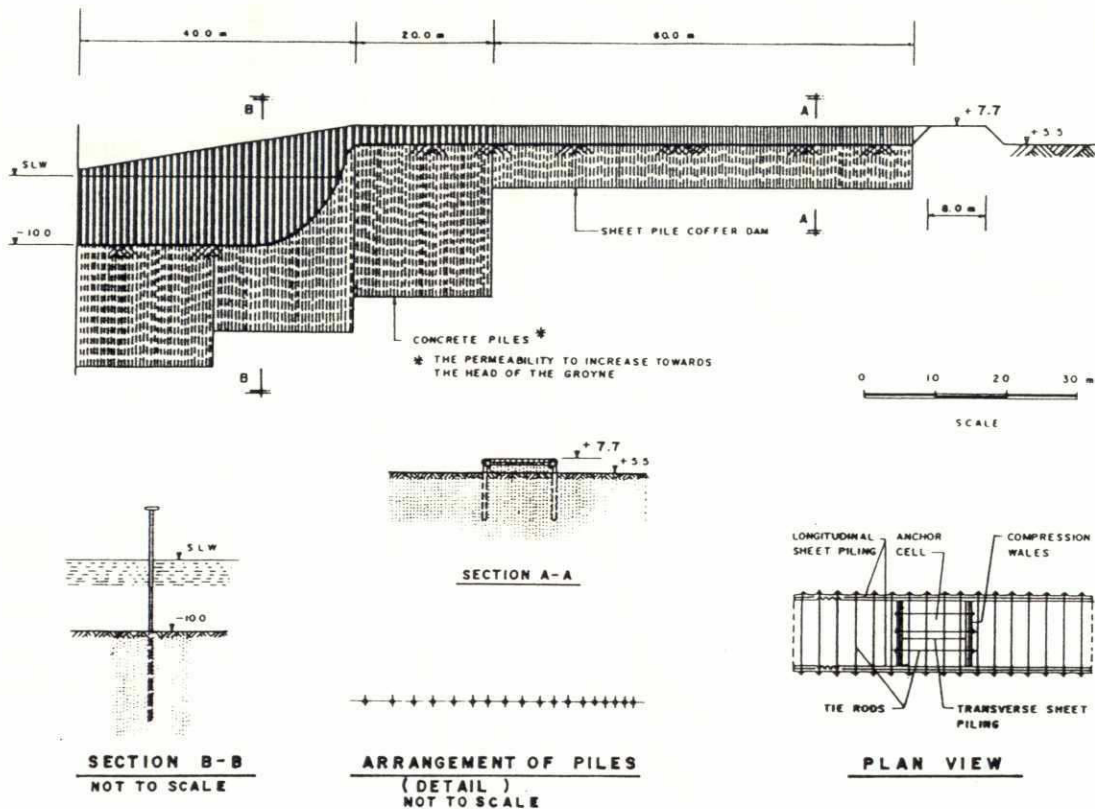
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FIG. A.20-8

TYPE GA-II



TYPE GA-12

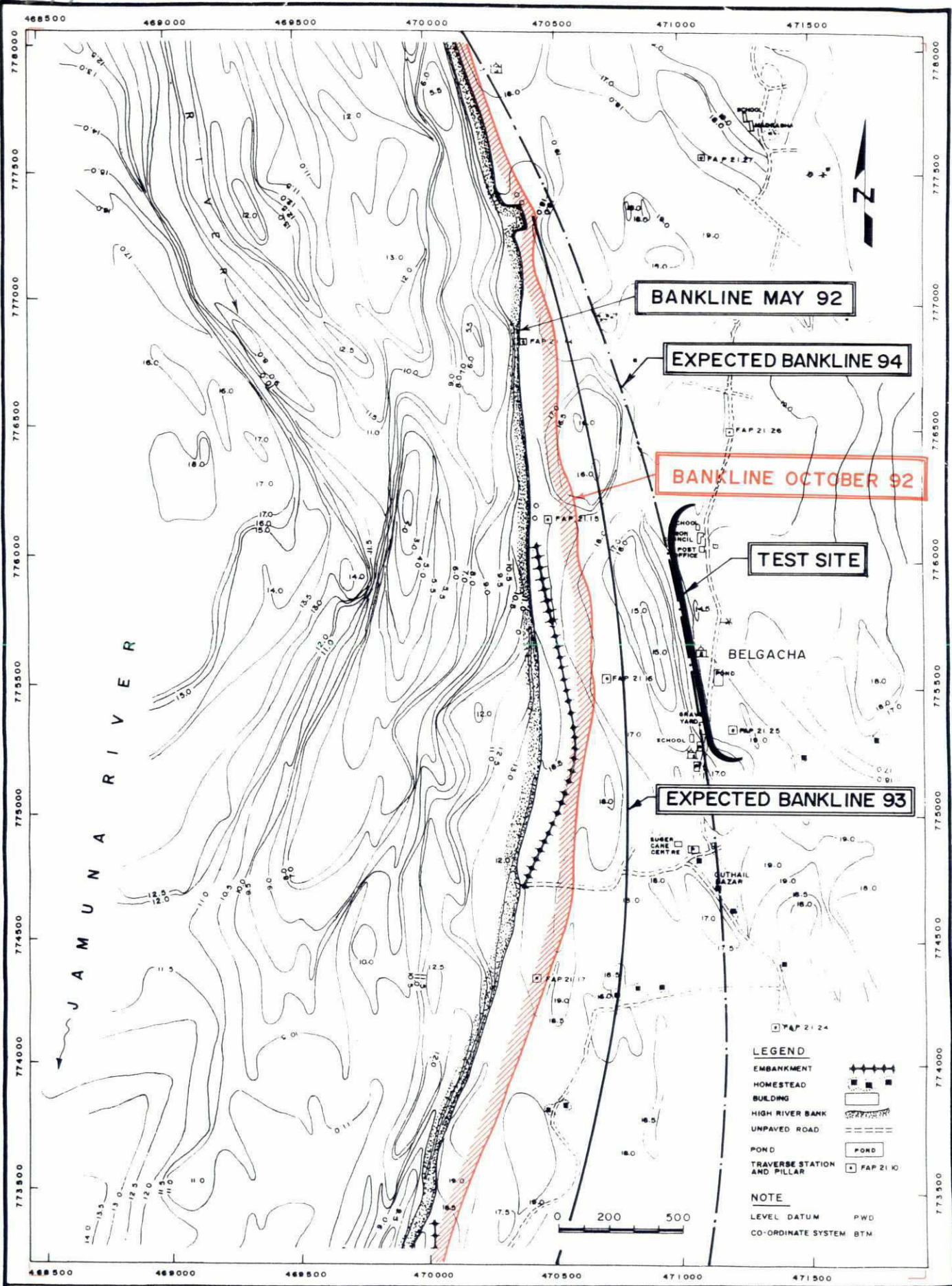


ALTERNATIVE GROYNES AT KAMARJANI TEST SITE

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FIG. A.20-9



LOCATION OF TEST SITE AT BAHADURABAD

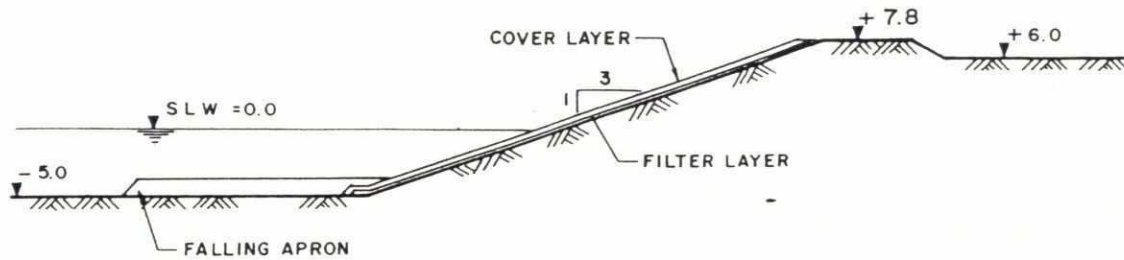
FAP - 21/22
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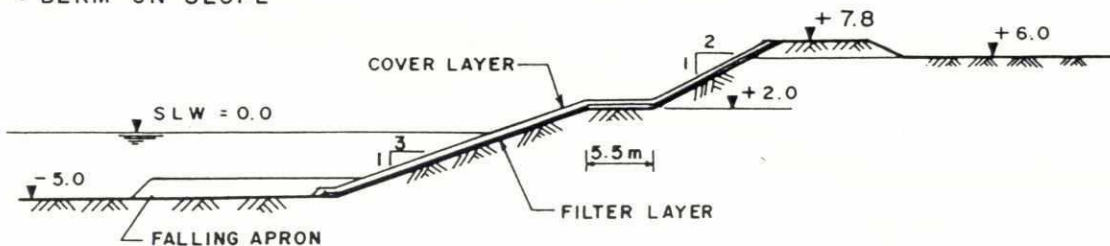
FIG A.20-10

TYPE BA-1

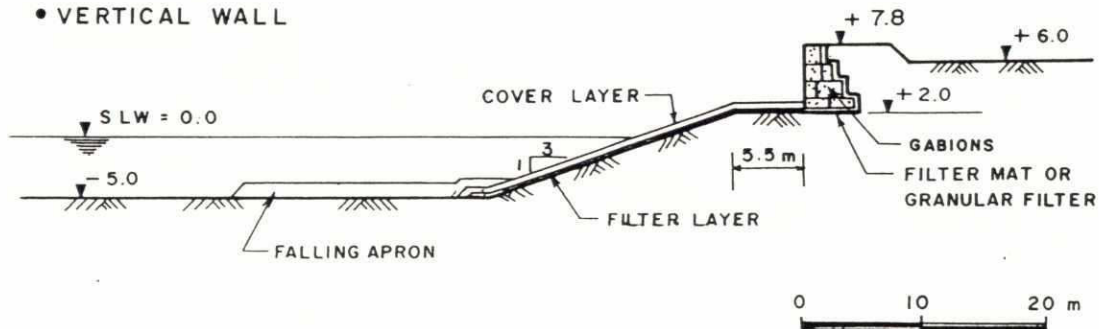
- SHALLOW FALLING APRON
- NO BERM ON SLOPE

TYPE BA-2

- SHALLOW FALLING APRON
- BERM ON SLOPE

TYPE BA-3

- SHALLOW FALLING APRON
- VERTICAL WALL



CROSS SECTIONS FOR EMBANKMENT
AT BAHADURABAD
TEST SITE

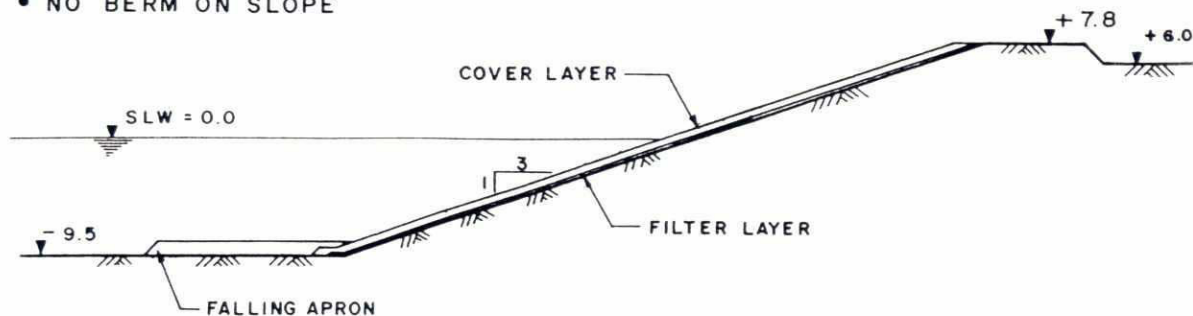
FAP - 21/22
BANK PROTECTION AND RIVER TRAINING /
(AFPM) PILOT PROJECT

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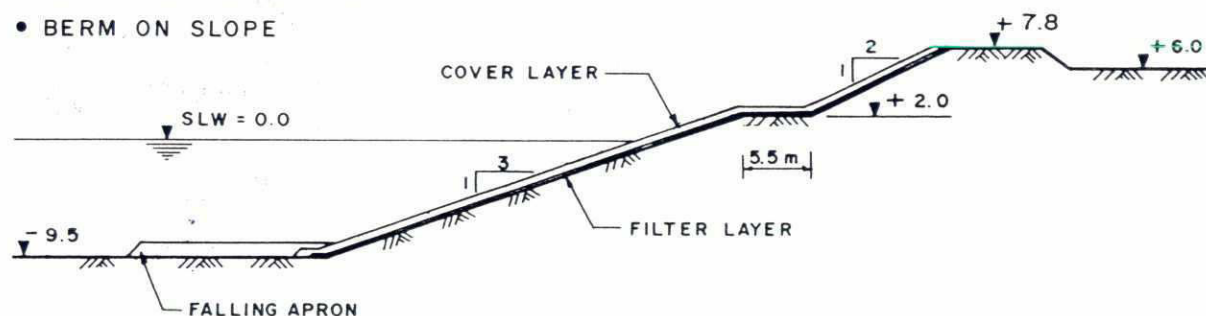
FIG. A.20-11

TYPE BA - 4

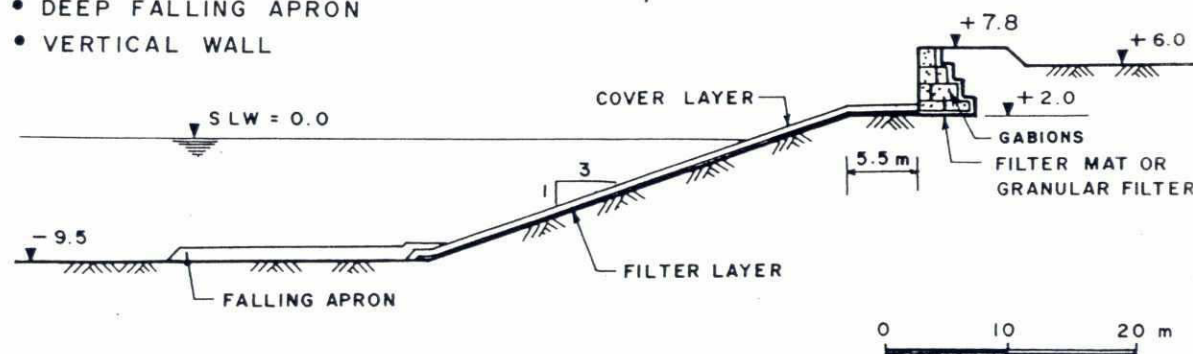
- DEEP FALLING APRON
- NO BERM ON SLOPE

**TYPE BA - 5**

- DEEP FALLING APRON
- BERM ON SLOPE

**TYPE BA - 6**

- DEEP FALLING APRON
- VERTICAL WALL



**CROSS SECTIONS FOR EMBANKMENT
AT BAHADURABAD
TEST SITE**

FAP - 21/22

BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT

FINAL REPORT

FIG. A.20-12

**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 21

**PRELIMINARY DESIGN OF TEST
STRUCTURES**

JANUARY 1993

ANNEX 21

PRELIMINARY DESIGN OF TEST STRUCTURES

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1 INTRODUCTION

Based on morphological studies several possible test areas have been investigated both on the right and on the left bank of the Jamuna out of which finally 2 sites have been selected for the construction of test structures (see Section 8.1 and 8.2 of the Main Report) viz Kamarjani and Bahadurabad (see Fig.A.21-1).

Kamarjani is situated on the right bank of the Jamuna, about 15 km downstream of the confluence of the Teesta river. The area experiences serious bank erosion which is likely to continue over the coming years. Hence the probability of the test structures to be exposed to flow attack until the end of the Test and Implementation Phase is rather high. The change of the bankline during the monsoon season of 1992 is shown on Fig.A.21-2 as is the selected test site, located about 3 km north of the Manos regulator.

The test site Bahadurabad is situated about 3.5 km south of Bahadurabad ghat on the left bank of the Jamuna in an area with a fair probability for a continuous and sufficient flow attack during the Test and Implementation Phase. The test site and the changes of the bankline from May to October 1992 are shown on Fig.A.21-11.

At Kamarjani, it is planned to construct in 1993/94 a field of 5 groynes, each consisting of an impermeable stretch on the flood plain and a permeable pile structure in the river. The construction period at Bahadurabad site is scheduled for 1994/95 where a number of slope revetments will be built which are different in composition, dimensions, construction materials, construction methods and bed protection.

2 PRELIMINARY DESIGN OF GROYNES

2.1 LAYOUT OF TEST STRUCTURES

One of the objectives to be achieved by FAP 21 is to develop and optimize design criteria for bank protection in the form of groynes. Since there is, however, no rule of general applicability for dimensioning of groynes depending on a number of different design parameters, physical model tests had been performed in order to define the optimum alignment and layout of the test structures, to determine the flow velocities in their vicinity, the location of scour holes and their maximum depth.

A stretch of the river bank of a certain length can only be sufficiently protected by a series of groynes and not by a single groyne only. Hence a field of 3 groynes was tested in a number of runs in the model tests with variations of the type of groynes, their length and spacing and their orientation to the current. One of the most important results of these investigations was to realize that the scour depth at the head of permeable groynes is only

in the order of 6 m compared with 20 to 30 m at the head of impermeable groynes. This phenomenon could be influenced by gradually increasing the spacing of piles of the permeable structures.

Based on the results of the physical model tests it was decided to construct a series of 5 groynes at Kamarjani with an orientation of 15° to the flow direction. The total length of the groynes will be 120 m and the spacing 250 m (see Fig.A.21-3). Since the scour depth statistically to be expected at the groyne head during the Test Phase duration is in the order of about 6 m only, the provision of a bed protection is not necessarily required, will however be tested using different dimensions and various materials.

2.2 TYPE OF STRUCTURES

Out of the alternative design solutions discussed in ANNEX 20 viz

- impermeable (solid) groynes
- permeable (open) groynes and
- a combination of impermeable and permeable stretches

the former are to be ruled out with regard to the deep scour holes at their head.

Finally selected as test structures were the combinations of impermeable stretches on the flood plain built as a conventional earth dam or as a sheet pile cofferdam and permeable stretches built as pile structures. The permeable sections, starting with about 10 % permeability and increasing to about 80 % permeability at the heads will be built of steel piles. In the study of alternative design solutions (see ANNEX 20) also the use of concrete piles has been taken into consideration. First structural calculations have shown, however, that probably solid piles 650 mm would be required. With regard to their weight and the heavy equipment needed to handle them on site, their use is ruled out at this stage of the Project. However, more detailed investigation and calculations will be carried out in the Final Design Phase to develop, if possible, practicable solutions for the use of concrete/prestressed concrete piles. If found to be feasible, one or two of the permeable stretches of the groynes would be built of concrete/prestressed concrete piles instead of the now proposed steel piles. Even for the steel piles relatively large diameters are required to resist the current induced forces in case the top of the piles should reach above the design water level i.e. to + 7.7 m PWD. Therefore the Consultant suggests to reduce the length of the piles and to allow them to be submerged at high water stages. Thus out of the 12 alternatives presented and discussed in ANNEX 20 only the following seem to be feasible:

- Type GA-4: . impermeable stretch on flood plain: earth dam
- . permeable stretch in the river: steel piles, partially submerged at high water stages
- Type GA-10: . impermeable stretch on flood plain: sheet pile cofferdam
- . permeable stretch in the river: steel piles partially submerged at high water stages

Hence, the variations that will be investigated are:

- 1 impermeable stretch
 - earth dam
 - sheet pile cofferdam
- 2 permeable stretch
 - steel piles
 - (if possible, concrete piles)
- 3 scour protection material
 - provision
 - no provision

The following variations of system have been selected (see Fig.A.21-4):

Test No.	Type No.	Impermeable/permeable stretch	bed protection
A	GA-10	steel sheet pile cofferdam/steel piles	concrete blocks
B	GA-10	concrete sheet pile cofferdam/steel (concrete) piles	sand bags
C	GA-4	earth dam/steel piles	chemically bonded
D	GA-4	earth dam/steel piles	none
E	GA-4	earth dam/steel (concrete) piles	sand blocks

Details of the various test structures are shown in Fig.A.21-5 through A.21-9.

On top of the permeable pile structure a gangway will be constructed to allow monitoring and servicing, including small repair or emergency dumping of blocks or sand bags for scour protection.

2.3 PRELIMINARY DESIGN OF COMPONENTS

2.3.1 Design Criteria

Based on the results of topographical survey, hydrographical survey, hydrometric measurements, physical and mathematical model tests the following design criteria for the test structures at Kamarjani were fixed (for details see Section 7.2 and 8.3 of the Main Report)

(i) **Water levels:**

- standard low water level : + 15.2 m PWD
- standard high water level : + 20.7 m PWD
- high flood level : + 23.3 m PWD ($\cong$ SLW + 8.1 m)
(100 years return period)
- design water level : SLW + 7.7 m
(25 years return period)

(ii) Soil conditions:

- flood plain level : + 20.5 to 21.0 m PWD
- based on the results of : $\gamma/\gamma' = 18/8 \text{ kN/m}^3$
 boring No. GA 2 and $\varphi' = 27.5^\circ$
 GA 4 the following soil $c' = 0 \text{ kN/m}^2$
 properties can be assumed
- relative density: down to
 about + 5 m PWD : loose
 below ± 0 m PWD : medium dense to dense
- groundwater level : about + 17.5 m PWD

(iii) Hydraulic loads

- design velocity : $u = 2.5 \text{ m/s}$
- significant wave height : $H_s = 1.0 \text{ m}$
- wave period : $T = 3 \text{ s.}$

(iv) Scour depth:

- at the head of permeable : 6 m below river bed
 groynes

The greater depths are to be expected at the first groyne of a series, the lower values at the following ones.

(v) Earthquake loads:

Earthquake loads and liquefaction of soil are not considered, since their frequency of occurrence is rather low and cannot be expected during the Project period. Hence the test structures would be "overdesigned" which is not according to the design philosophy that requires a lower level of safety to allow a certain damage of the structures.

2.3.2 Earth Dam

The earth dam being the impermeable stretch on the flood plain over a length of 60 m can be built of hydraulic fill with 1:3 slopes. The crest width is proposed to be 8 m. A protection layer on the crest surface is not necessarily required. Hence natural protection with grass is recommended.

The following protection measures of the slopes are proposed:

(i) Test No. C:

(a) Cover layer:

- material : rip-rap with boulders with
 $D_{50} = 0.18 \text{ m} : 0.30 \text{ m}$
- thickness of layer : 0.30 m
- grading of boulders : $W_{15} = 6 \text{ to } 15 \text{ kg}$
 $W_{50} = 15 \text{ to } 23 \text{ kg}$
 $W_{100} = 30 \text{ to } 61 \text{ kg}$

- (b) filter layer :** Granular filter
 number of layers and their composition depends on the fill material and will be defined in the final design.

- (c) toe protection :** at the upstream and downstream flank of the dam only, with a length of 30 m starting at the transition from the impermeable to the permeable stretch.
 width: 5 m
 material: boulders (same as above)
 layer thickness: 0.60 m
 grading: same as above.

(ii) Test No. D:

(a) cover layer:

- material : rip-rap with boulders and bitumen grout
- thickness of layer : same as under Test No. C
- grading of boulders : same as under Test No. C

- (b) filter layer :** lean sand asphalt
 the exact composition is to be determined in the final design. The bitumen content, however, should be about 4 % and the layer thickness at least 15 cm.

- (c) toe protection :** none

(iii) Test No. E:

(a) cover layer:

- material : chemically bonded sand blocks
- thickness of layer : 0.25 m

- (b) filter layer :** granular filter

number of layers and their composition depends on the fill material and will be defined in the final design.

(c) toe protection : area to be protected same as under Test No. C.
material: chemically bounded sand blocks
layer thickness: 0.25 m.

2.3.3 Sheet Pile Cofferdam

The parallel steel sheet piling walls with a center-to-center distance of 8 m are to be driven or sunk by appropriate means. The two walls must be connected by tie rods the distance of which depends on the statical requirements. For safety reasons it is recommended to install transverse walls and anchor cells which would confine any damage to that section of the cofferdam in which the damage occurs.

The thickness and the total length of the sheet piles are to be determined in the final design just as the diameter and center-to-center distance of the tie rods taking into consideration material being available in Bangladesh. The fill material may be hydraulic fill the surface of which is to be protected by provision of grass. A toe protection is normally not required because the embedded length of the sheet piles can be increased accordingly. However, for safety reasons it is recommended to provide at the upstream side and over a length of 30 m of the downstream side of Test No. A a falling apron with a width of 5.0 m. The layer thickness, the material and its grading should be the same as described under Test No. C.

Alternatively the use of prestressed concrete sheet piles seems to be feasible. Since, however, their length available in Bangladesh is limited to 6.0 m, the final decision on their application depends on the results of the final design. In this case, however, a toe protection would be compulsory over a length of starting at the transition from the permeable to the impermeable stretch with regard to the limited embedded length of the sheet piles.

2.3.4 Steel Piles

As a result of the investigations for this study it was found out that spiral welded piles of a maximum diameter 219.1 mm with a maximum wall thickness of 8.18 mm are readily available in the market in Bangladesh. Structural calculations, however, have shown that these piles are not sufficient for the given design criteria when used in a single or double row. Only combinations of 3 or 4 piles would be strong to withstand the current and wave induced loads. The maximum diameter of single piles arranged in a double row and their tops at about 2.0 m above SLW, is in the order of 500 mm with a wall thickness of 10 mm. Piles of this type can be manufactured in Bangladesh. The spacing between the 2 rows will be about 1.5 m and the individual piles are to be braced to each other by traverses and diagonals.

2.3.5 Concrete Piles

The maximum in Bangladesh produced concrete piles have a diameter of 650 mm and a wall thickness of 80 mm. This size of pile is not sufficient to be used in a single or double row within the permeable stretch of the groynes. Solid prestressed concrete piles would be too heavy for handling and will not be considered. Thus detailed calculations are necessary in the Final Design Phase to develop possibly practical solutions for the use of concrete/prestressed concrete piles.

2.3.6 Bed Protection

The results of the physical model tests showed that scour holes at the head of permeable groynes are to be expected in the magnitude of 3 to 6 m only. The provision of bed protection in the form of falling aprons would therefore not be necessarily required, they will however be tested. The dimensions of the falling apron in the area of the open pile structures are shown in Fig. A.21-10.

The following material is planned to be used:

Test No. A: 1.0 m thick layer of concrete blocks $D = 0.40$ m

Test No. B: 1.2 m thick layer of partially filled sand bags, size $1.0 \times 1.0 \times 0.5$ m

Test No. C: 0.6 m thick layer of chemically bonded sand blocks.

Test No. E: same as Test No. B, layer thickness however, 0.8 m only

3 CONSTRUCTION METHODS OF GROYNES

The designed groynes consist of an impermeable part to be built on the flood plain either as earth dam and or as sheet pile cofferdam and of a permeable stretch in the form of pile structures.

The material needed for the construction of the earth dam and also for filling of the sheet pile cofferdam may be taken from the flood plain areas between the groynes. This may be done by hand or mechanical means, whereas the compaction of the fill material must be executed in any case by mechanical means. The fill shall be placed in layers and sufficiently compacted in order to avoid settlements due to consolidation. The type of compactor depends on the layer thickness and on the type of soil used.

Placing of filter layers being granular material and lean sand asphalt respectively can easily be executed in the dry and requires no special experience. The same holds for the installation of the cover layers on the slopes viz rip-rap of boulders, in one case grouted with bitumen and chemically bonded sand blocks.

Driving of sheet piles for the cofferdams can be executed from the flood plain whereas driving/sinking of piles for the permeable section of the groynes requires more or less heavy pile driving equipment which must be pontoon mounted. Floating equipment is in any case necessary, also in case the piles will be sunk with the help of water jetting or vibration.

The material for the bed protection around the pile structures is to be dumped from barges upto to the designed thickness. For more details regarding constructional aspects see ANNEX 19.

4 PRELIMINARY DESIGN OF REVETMENTS

4.1 LAYOUT OF TEST STRUCTURES

The bank revetment test structures are proposed to be built west of the village Belgacha (see Fig.A.21-11). The alignment will be straight with termination points at the ends which must be carefully secured in order to avoid outflanking of the structure. Particularly the upstream termination structure will have to be protected as heavy as the main river bank. For cost reasons all different types of structures will be arranged in one line to avoid double termination points. The exact location of the test structure has to be determined after the monsoon season of 1993, since the implementation of works is planned for 1994 only and the morphological changes are uncertain taking into account that all predictions made are to be considered as tentative only.

The total length of the structure must be limited to about 1 km not only with respect to available funds but also to the tight time frame for the execution of works during the dry season from November 1994 to May 1995. Hence only a construction period of about 8 months is available which would not allow the construction of a greater length of revetments especially taking into account the constructional aspects involved due to the different cross-section designs.

A very important detail to be investigated is the toe protection of the revetment in the form of a falling apron. It is planned to test and to monitor the behaviour of the falling apron of different dimensions using various materials at different water depths viz 5.0 m and 9.5 m respectively below SLW.

Taking into account the presently prevailing topographical and hydrographic conditions at the test site, 6 cross-sections of revetments seem to be feasible (see Fig.A.20-11 and A.20-12 of ANNEX 20) out of which 4 nos. are selected for execution with regard to the time frame and taking into consideration constructional aspects. These are:

Type BA-1: . shallow falling apron
 . no berm on the slope

- Type BA-2: . shallow falling apron
 . berm on the slope
 Type BA-5: . deep falling apron
 . berm on the slope
 Type BA-6: . deep falling apron
 . vertical wall

Thus the distinctive features of bank revetments are covered and details are shown on Fig.A.21-12. Further variations are arising through the application of various materials, see further below.

For testing of the different materials and systems of cover layer, filter layer and toe protection the above mentioned types of cross-sections have to be arranged in a reasonable sequence avoiding that one failing design would cause the failure of the next one. Therefore it is intended to start at the upstream termination with the water depth of 9.5 m below SLW leaving the construction in shallow water for the second half of the revetment further downstream. The layout and sequence of cross-sections is shown in Fig.A.21-13.

4.2 TYPE OF STRUCTURES

Alternative design solutions of revetments consisting in each case of the three components

- cover layer;
- filter layer, and
- toe protection

have been studied in ANNEX 20 to this report, taking into consideration the prevailing conditions and their possible changes till the implementation of works at the selected test site in 1994. After carefully sorting out all construction methods and materials that are not appropriate for becoming potential standard solutions on the Jamuna because they are either technically not fitting the requirements or involving too demanding construction techniques or too expensive materials that could be replaced by cheaper local ones, the following systems were selected to be applied on the test structures.

1. above low water level/ground water level

(a) Vertical Wall

V 2	wire mesh gabions filled with bricks
V 3	wire mesh gabions filled with boulders

(b) Coverlayer

A 1	rip-rap with stones or boulders
A 2	rip-rap chemically bonded sand blocks
A 3	rip-rap with cement grout
A 4	rip-rap with bitumen grout
A 6	concrete blocks, regularly placed
A 8	open stone asphalt
A 10	wire mesh gabions filled with bricks
A 11	wire mesh gabions filled with boulders

(c) **Filter layer**

FA 1	Granular filter
FA 2	lean sand asphalt
FA 3	geotextile filter (one layer)
FA 4	geotextile filter (multi-layer)

2. below low water level/ground water level(a) **Coverlayer**

B 1	rip-rap with boulders
B 2	rip-rap chemically bonded sand blocks
B 5	concrete blocks
B 6	cable connected blocks
B 7	blocks connected to geotextiles
B 9	geotextile bags filled with sand/lean sand asphalt
B 10	tubular fabrics filled with sand/lean sand asphalt
B 12	wire mesh mattresses filled with stones/boulders

(b) **Filter layer**

FB 1	FB 1 geotextile filter (one layer)
FB 2	FB 2 geotextile filter (multi layer)
FB 3	FB 3 geotextile - sand mat

(c) **Toe protection**

T 1	boulders
T 2	concrete blocks
T 3	geotextile bags, partially filled with sand
T 4	cable connected blocks as extension of revetment
T 5	tubular fabrics filled with sand or lean sand asphalt as extension of revetment
T 6	wire mesh mattresses filled with stones/bricks
T 7	scour prevention mat

The material needed for the construction of the above listed systems was found to be either available in Bangladesh or can be procured. Further to conventional material like boulders and concrete blocks also chemically bonded blocks shall be tested as developed in China and production of which is planned to be taken up in Bangladesh in March 1993 (details see ANNEX 19). With regard to the filter layers some innovative composite material of geosynthetics, geonaturals (jute) and minerals will be used and the tests will refer to placing techniques and technical performance of the material.

The cross-sections selected in Section 4.1 are to be protected by different combinations of cover layer, filter layer and toe protection. Out of the sheer endless number of possible variations the following have been chosen (see Table A.21/4-1):

Test No.	Cross-section No.	Cover layer/filter layer		Toe Protection
		Above water level	Below water level	
A	BA-6	V3 : Wire mesh gabions (boulders) A1 : Rip-rap (boulders) FA4: geotextile filter (multi layer)	B1 : rip-rap (boulders) FB3: geotextile sand mat	T1: boulders
B	BA-6	V2 : wire mesh gabions (bricks) A2 : rip-rap (chemically bonded sand blocks) FA4: geotextile filter (multi layer)	B2 : rip-rap (chemically bonded sand blocks) FB3: geotextile sand mat	T3: geotextile bags, partially filled with sand
C	BA-5	A4 : rip-rap (boulders with bitumen grout) FA1: granular filter	B9 : geotextile bags filled with sand FB1: geotextile filter (one layer)	T3: geotextile bags, partially filled with sand
D	BA-5	A6 : concrete blocks (regularly placed) FA4: geotextile filter (multi layer)	B5 : concrete blocks FB3: geotextile sand mat	T2: concrete blocks
E	BA-5	A3 : Rip-rap with cement grout FA2: lean sand asphalt	B5 : concrete blocks FB2: geotextile filter (multi layer)	T2: concrete blocks
F	BA-2	A8 : open stone asphalt FA2: lean sand asphalt	B10: tubular fabrics filled with sand/ lean sand asphalt FB1: geotextile filter (one layer)	T5: tubular fabrics filled with sand or lean sand asphalt as extension of revetment
G	BA-2	A11: wire mesh gabions filled with boulders FA3: geotextile filter (one layer)	B12: wire mesh mattresses filled with boulders FB2: geotextile filter (multi layer)	T6: wire mesh mattresses filled with boulders
H	BA-1	B6 : cable connected blocks FA4: geotextile filter (multi-layer)	B6 : cable connected blocks FB3: geotextile sand mat	T4: cable connected blocks as extension of revetment
K	BA-1	B7 : blocks connected to geotextiles FA3: geotextile filter (one layer)	B7 : blocks connected to geotextiles FB1: geotextile filter (one layer)	T7: scour prevention mat

Table A.21/4-1: Revetment composition selected for test structures

4.3 PRELIMINARY DESIGN OF COMPONENTS

4.3.1 Design Criteria

Based on the results of topographical survey, hydrographical survey, hydrometric measurements, physical and mathematical model tests the following design criteria for the test structures at Bahadurabad were fixed (for details see Section 7.2 and 8.3 of the Main

Report):

(i) **Water levels**

- standard low water level : + 13.3 m PWD
- standard high water level : + 18.8 m PWD
- high flood level : + 21.4 m PWD ($\hat{=}$ SLW + 8.1 m)
(100 years return period)
- design water level : SLW + 7.8 m
(25 years return period)

(ii) **Soil conditions**

- flood plain level : + 18.0 to 19.0 m PWD
- based on the results of : $\gamma/\gamma' = 18/8 \text{ kN/m}^3$
boring No. BA-3 and BA-4 $\phi' = 27.5^\circ$
the following soil properties $c' = 0 \text{ kN/m}^2$
can be assumed
- relative density:
down to about +13.0 m PWD: loose
down to about ± 0.0 m PWD: medium dense
below ± 0.0 m PWD: dense
- groundwater level : + 15.0 m PWD.

(iii) **Hydraulic loads**

- design velocity : $u = 3.5 \text{ m/s}$
- significant wave height : $H_s = 1.0 \text{ m}$
- wave period : $T = 3 \text{ s}$

(iv) **Scour depth:**

- up to 11 m below river bed at the upstream termination point
- about 5 m below river bed further downstream

(v) **Earthquake loads:**

Earthquake loads and liquefaction of soil is not considered, since the frequency of occurrence is rather low and cannot be expected during the Project period. Hence the test structures would be "overdesigned" which is not according to the design philosophy that requires a lower level of safety to allow a certain damage of the structures.

4.3.2 Slope

Based on extensive slip circle calculations which took account the soil conditions and water levels as given in Subsection 4.3.1, the underwater slope is recommended to be executed 1:3 (vertical: horizontal) and the slope above the water level in 1:2.

4.3.3 Filter Layer

(1) General

The function of a filter layer is separation of the cover layer from the subsoil, prevention of migration of subsoil particles out of the bank slope allowing, however, at the same time movement of water through the filter.

The filter criterion relates the grading of the filter to that of the subsoil. According to the results of boring No. BA-3 and BA-4 the subsoil at the test site consists of cohesive layers of clayey sandy silt to sandy silt down to about 5.5 m followed by poorly graded silty sand. Grain size distribution curves determined on soil samples of both layers are shown in Fig.A.21-14.

The filter layer may consist of granular or fibre filters taking into account, however, that a controllable construction of granular filters below the water level is not possible, especially not in flowing water. Hence the provision of both filter types is feasible, granular filters above and fibre filters below the low water and groundwater level respectively. Alternatively the construction of a filter consisting of lean sand asphalt is possible.

(2) Granular filter

The granular filter is to be designed in accordance with relevant filter rules given in ANNEX 17. Decisive is the left border of the grain size distribution plot of Fig.A.21-14 (a), from which the following particle sizes can be taken:

$$d_{15} = 0.004 \text{ mm}; \quad d_{50} = 0.015 \text{ mm}; \quad d_{60} = 0.019 \text{ mm}; \quad d_{85} = 0.045 \text{ mm}.$$

In order to fulfil the permeability and retention criteria 3 separate filter layers are required to ensure a satisfactory performance in the long run. The following layers are selected and shown in Fig.A.21-14 (c):

$$\text{1st filter layer: } D_{10} = 0.08 \text{ mm}; \quad D_{60} = 0.03 \text{ mm}; \quad U = 4;$$

$$\text{2nd filter layer: } D_{10} = 1.4 \text{ mm}; \quad D_{60} = 4.2 \text{ mm}; \quad U = 3;$$

$$\text{3rd filter layer: } D_{10} = 20 \text{ mm}; \quad D_{60} = 60 \text{ mm}; \quad U = 3;$$

The minimum thickness of layer No. 1 and 2 is 15 cm and of the 3rd layer 30 cm.

(3) Lean sand asphalt filter

The exact composition of the lean sand asphalt filter to be installed in test No. E and F is to be determined in the final design. The bitumen content should, however, be about 4 % and the layer thickness at least 15 cm.

(4) Geotextile filters

The filter must have a porous structure where the combination of fibres and pores allows the particles of the soil to be retained in such a way that no blocking or clogging of the filter can occur. The amount of pores of different geotextiles is as follows:

- needle-punched non-woven geotextiles	:	90 %
- thermally bounded non-woven geotextiles	:	70 %
- woven geotextiles	:	20 %

The above shows that needle-punched geotextiles provide the largest pore-fibre ratio and are hence recommended to be used since they are also offering striking advantages in terms of resistance to puncturing.

To design a geotextile for the soil to be retained, its effective opening size must be appropriate to this soil. The grain sizes of the top soil layer are as follows:

$$d_{10} = 0.0025 \text{ mm}; \quad d_{30} = 0.015 \text{ mm}; \quad d_{60} = 0.019 \text{ mm}; \quad d_{90} = 0.056 \text{ mm}$$

The permeability is assumed to be $k_s = 10^{-6} \text{ m/s}$. Since more than 40 % of the soil particles are smaller than 0.06 mm (see Fig.A.21-14 (a)), the soil is to be classified according to PIANC (1987a) (see ANNEX 17) as range A. The following retention criteria are to be fulfilled:

$$\begin{aligned} O_{90} < d_{90} &= 0.056 \text{ mm} \\ < 10 * d_{50} &= 0.15 \text{ mm} \\ < 0.3 \text{ mm} \end{aligned}$$

Thus the filter fabric $O_{90} < 0.056 \text{ mm}$

The permeability of the geotextile must be at least 50 times more than that of the soil, Thus $K_g > 50.k_s = 5.10^{-5} \text{ m/s}$.

Since the soil was identified to be a problem soil susceptible to downslope migration, a stabilizing thick layer of coarse fibers attached to the back of the geotextile filter is recommended. This must have according to PIANC (1987a) an effective opening size in the range of:

$$0.3 < O_{90} < 1.5 \text{ mm}$$

The thickness of the layer should be

$$5 < t_{gg} < 15 \text{ mm}.$$

The grain sizes of the second layer are (see Fig. A.21-14 (b))

$$d_{10} = 0.04 \text{ mm}; \quad d_{30} = 0.09 \text{ mm}; \quad d_{60} = 0.010 \text{ mm}; \quad d_{90} = 0.017 \text{ mm}$$

The permeability was determined to be $k_s = 3.10^{-5} \text{ m/s}$.

This soil is to be classified as range B/C.

Thus the following effective opening size is required:

$$\begin{aligned} O_{90} &< 1.5 \cdot 0.04 \sqrt{3.5} = 0.11 \text{ mm} \\ &< 10 \cdot d_{50} = 0.9 \text{ mm} \\ &< 0.3 \text{ mm} \end{aligned}$$

The required permeability of the geotextile is

$$k_g > 50 \cdot k_s = 50 \cdot 3 \cdot 10^{-5} = 1.5 \cdot 10^{-4} \text{ m/s}$$

Because the coefficient of uniformity is smaller than 15, also this soil is susceptible to downslope migration. Hence a coarse layer attached to the back of the geotextile is also recommended in this case:

$$\begin{aligned} 0.5 &< O_{90} &< 2.0 \text{ mm} \\ 5 &< t_{gg} &< 20 \text{ mm.} \end{aligned}$$

The minimum thickness of both geotextile filters should be 6 mm.

4.3.4 Cover Layers

Based on the equations discussed and explained in ANNEX 16 the size/thickness of cover layer materials have been estimated taking into account the different slopes of 1:2 and 1:3 above and below the water level respectively.

(a) Slope 1:2 above water

A 1	rip-rap with stones or boulders	: $D_{50} = 0.40 \text{ m}$
A 2	rip-rap chemically bonded sand blocks	: *
A 3	rip-rap with cement grout	: $D_{50} = 0.36 \text{ m}$
A 4	rip-rap with bitumen grout	: $D_{50} = 0.36 \text{ m}$
A 6	concrete blocks, regularly placed	: $D = 0.45 \text{ m}$
A 8	open stone asphalt	: $d = 0.30 \text{ m}^{**}$
A 10	wire mesh gabions filled with bricks	: $D = 0.20 \text{ m}$; $d = 0.36 \text{ m}^{**}$
A 11	wire mesh gabions filled with boulders	: $D = 0.20 \text{ m}$; $d = 0.36 \text{ m}^{**}$

* The largest dimensions of blocks made until now are $0.4 \cdot 0.4 \cdot 0.25 \text{ m}$ only. In case it will turn out that the production of blocks of larger dimensions, say at least cubes of 0.45 m size, cannot be realized, it is planned to fix two blocks of the above size together by appropriate means.

** d means layer thickness and height of gabions/mattresses respectively.

The weight of boulders under A 1 should be, according to equation (4-1) and Fig.4.1 of ANNEX 16, in the following ranges:

$$\begin{aligned} W_{15} &= 60 \text{ to } 160 \text{ kg} \\ W_{50} &= 160 \text{ to } 250 \text{ kg} \\ W_{100} &= 330 \text{ to } 660 \text{ kg.} \end{aligned}$$

In case the boulders are grouted with bitumen or mortar, the following grading would be sufficient:

$$\begin{aligned} W_{15} &= 50 \text{ to } 120 \text{ kg} \\ W_{50} &= 120 \text{ to } 180 \text{ kg} \\ W_{100} &= 240 \text{ to } 480 \text{ kg.} \end{aligned}$$

(b) Slope 1:3 below water

B 1	rip-rap with boulders	: $D_{50} = 0.48 \text{ m}$
B 2	rip-rap chemically bonded sand blocks	*
B 5	concrete blocks	: $D = 0.40 \text{ m}$
B 6	cable connected blocks	: $D = 0.25 \text{ m}$
B 7	blocks connected to geotextiles	: $D = 0.35 \text{ m}$
B 9	geotextile bags filled with sand/lean sand asphalt	: size of bags: $1.0 \times 1.0 \times 0.5 \text{ m}$
B 10	tubular fabrics filled with sand/lean sand asphalt	: $d = 0.35 \text{ m}^{**}$
B 12	wire mesh mattresses filled with stones/boulders	: $D = 0.17 \text{ m}$; $d = 0.30 \text{ m}^{**}$

(*) and (**) see under (a) above.

The following grading of boulders under B1 is required:

$$\begin{aligned} W_{15} &= 115 \text{ to } 285 \text{ kg} \\ W_{50} &= 285 \text{ to } 430 \text{ kg} \\ W_{100} &= 575 \text{ to } 1150 \text{ kg.} \end{aligned}$$

4.3.5 Vertical Wall of Wire Mesh Gabions

The individual wire mesh gabions should have a height of 1.0 m whereas the other dimensions could be larger. The diameter of stones should be minimum 0.20 m having a weight of about 21 kg.

4.3.6 Toe Protection

With reference to Subsection 7.4.6 of the Main Report the falling aprons as toe protection are designed as follows (compare also Table A.21/4-1):

Test No. A: material : boulders, grading as under B 1 above.

$$\begin{aligned} L &= 16.0 \text{ m} \\ T_i &= 1.5 \text{ m, } T_e = 1.5 \text{ m} \end{aligned}$$

Test No. B: material : geotextile bags 75 % filled with sand

$$\begin{aligned} L &= 16.0 \text{ m} \\ T_i &= 1.2 \text{ m, } T_e = 1.2 \text{ m} \end{aligned}$$

Test No. C: material: geotextile bags, 75 % filled with sand

$$\begin{aligned} L &= 16.0 \text{ m} \\ T_i &= 0.80 \text{ m}; T_e = 0.80 \text{ m} \end{aligned}$$

Test No. D: material : concrete blocks

$$\begin{aligned} L &= 16.0 \text{ m} \\ T_i &= 0.8 \text{ m}; T_e = 0.80 \text{ m (2 layers)} \end{aligned}$$

Test No. E: material : concrete blocks

$$\begin{aligned} L &= 16.0 \text{ m} \\ T_i &= 0.8 \text{ m}; T_e = 1.20 \text{ m (3 layers)} \end{aligned}$$

Test No. F: material : tubular fabrics filled with lean sand asphalt

$$\begin{aligned} L &= 10.0 \text{ m} \\ T_i &= T_e = D = 0.35 \text{ m (diameter as under B10)} \end{aligned}$$

Test No. G: material : wire mesh mattresses filled with boulders

$$\begin{aligned} L &= 16.0 \text{ m} \\ T_i &= 0.6 \text{ m}; T_e = 0.90 \text{ m} \end{aligned}$$

Test No. H: material : cable connected blocks

$$\begin{aligned} L &= 10.0 \text{ m} \\ T_i &= T_e = 0.25 \text{ m (as under B 6)} \end{aligned}$$

Test No. K: material : scour prevention mat

$$\begin{aligned} L &= 16.0 \text{ m} \\ \text{thickness} &: \text{to be specified in final design.} \end{aligned}$$

Legend:

$$\begin{aligned} L &= \text{length of falling apron} \\ T_i &= \text{initial thickness of falling apron at the toe of revetment} \\ T_e &= \text{thickness of falling apron at its end.} \end{aligned}$$



5 CONSTRUCTION METHODS OF REVETMENTS

The main idea of this project is to test the structures, in this case the revetments, and to monitor their behaviour under current and wave attack over a period of some years. Hence the exact execution of works in strict accordance with the design is of outstanding importance. This holds especially for placing the geotextile filters under water. Since this work might be critical if executed in flowing water, it is planned to build the first part of the test structures in stagnant water protected during their construction against waves and currents. Therefore a channel will be dredged from the river to the test site starting at the

downstream termination continuing upto the upstream termination. The remaining dam between the river and the test structures will protect the latter during the construction period. At the end of the construction time, a channel will be dredged towards the river which will be expanded by the current shortly washing the dam away. Thereafter the structure will be exposed to full flow and wave attack. Depending on the experiences made under stagnant water conditions and subject to having enough time available, it is intended to build the last tests under flowing water conditions.

Excavation and slope profiling for the revetments can be done from the landside or from the channel, using in the latter case pontoon mounted excavators, crawler dragline and cutter suction dredger. However, with regard to the very fine and uniform silty fine sand, the preparation of proper revetments slopes under water might be impossible. If so, the use of containment bunds is planned. The finished slope surface must be sufficiently smooth to ensure that the geotextile filter layer to be placed on it does not cover any larger voids.

The laying of geotextiles above water is a routine operation for experienced contractors, but also when they are placed under water, the fabrics must be correctly positioned, free of folds and with the proper overlapping. They must be kept in position until the cover layer has placed maintaining at the same time the lap zone free of ballast material. The fabric must be kept stable until the overlaying sheet is in position, and both sheets are to be ensured free of folds prior to placing the cover layer material. During placing of the cover layer any puncturing of the geotextile has to be strictly avoided.

In order to fulfil the above mentioned requirements, a geotextile system must have at the same time an appropriate rigidity and an adequate flexibility to facilitate the handling. Furthermore the complete system has to be sufficiently dense to remain in close contact with the subsoil while armouring is in progress.

The lapping difficulty is common to all methods of placing geotextiles and therefore the manufacturer of geotextiles should advise on the required lap width which should, however, not be less than 1.5 m in case of water depths up to 10 m at the time of placing.

The placing of cover layer material is described in more detail in ANNEX 19. The method to be applied depends on the type of material and system. In many cases the execution from the channel by means of pontoon mounted equipment is possible, especially as far as the underwater slope is concerned.

The falling apron material will be dumped in case of loose units such as boulders and concrete blocks. Geotextile bags and wire mesh mattresses are to be brought in position with more care to avoid damages during the installation process. Block mats and sand filled tubulars must be placed at the same time as the cover layer of the revetment since they are extensions of the latter. More details are given in ANNEX 19.

6 COST ESTIMATES

6.1 GROYNES

Based on the unit rates compiled in Section 9.3 of the Main Report, the total cost of the five test structures have been estimated as follows:

Test No.	Cost in Tk
Test No. A	42,398,940
Test No. B	36,269,840
Test No. C	33,997,190
Test No. D	27,740,130
Test No. E	33,655,840
Total Tk	174,404,940

The above costs are based on net working prices. They do not include costs for mobilization, site installation, contractor's overheads and profit, price escalation and contingencies. Furthermore the purchase of equipment is not included.

Details are shown in Table No. A.21.6-1 to A.21.6-5.

6.2 REVETMENTS

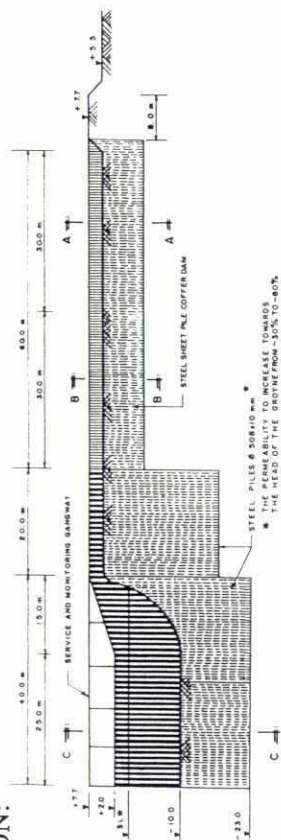
The total cost of the 9 revetments different in composition of cover layer, filter layer and toe protection have been estimated on the basis of the unit rates as per Section 9.3 of the main report as follows:

Test No.	Revetment (Tk)	Earth works (Tk)	Total Cost in Tk
Test No. A	14,513,550	7,262,100	21,775,650
Test No. B	13,330,550	7,262,100	20,592,650
Test No. C	12,924,500	7,700,800	20,625,300
Test No. D	14,693,000	7,700,800	22,393,800
Test No. E	15,716,400	7,700,800	23,417,200
Test No. F	5,241,500	4,988,150	10,229,650
Test No. G	10,765,310	4,988,150	15,753,460
Test No. H	8,824,000	4,749,000	13,573,000
Test No. K	10,061,000	4,749,000	14,810,000
Terminations	25,000,000	5,000,000	30,000,000
Total Tk	130,784,810	62,100,900	193,170,710

The above cost are based on net working prices including dredging, earth works and revetment works. They do not include costs for mobilization, site installation, contractor's overheads and profit, price escalation and contingencies.

Details are shown in Table No. A.21.6-6 to A.21.6-18.

DESCRIPTION:



On the flood plain : a) steel sheet pile coffer dam

b) bed protection upstream $l=60m$
downstream $l = 30m$, boulders

In the river : a) steel pile structure two rows dia 50cm, $t=10mm$
permeability increasing 30% to 80%

b) bed protection on both sides, CC-blocks

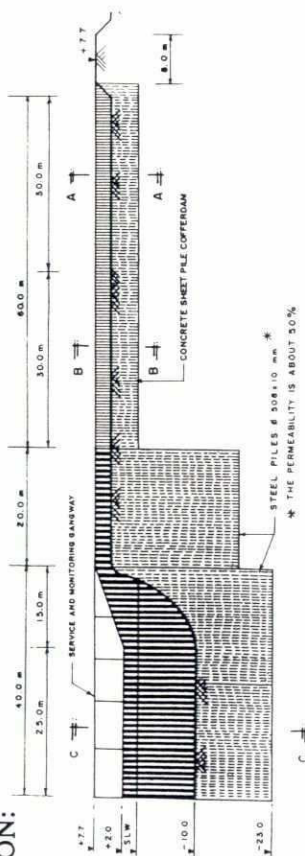
ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking steel sheet piles (82 kg/m ²), $t = 8 mm$	P6	m ²	4000	10	128	1280	40000	5,120,000
2	Bracing of steel or C.C. piles	P9	m	5000	1	128	128	5000	640,000
3	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	20	60	1200	700	42,000
4	Compaction with vibration plate	G7	m ³	10	20	60	1200	200	12,000
5	Supply, spreading & levelling of coarse sand $t = 0.15m$	AF1	m ²	140	8	60	480	1120	67,200
6	Herring bone bond bricks in 2 layer encased with two layer wire mesh	AP1	m ²	403	8	60	480	3224	193,440
7	Supplying & placing above water by 40-60kg. boulder ($t = 0.6m$) manually	AP5	m ²	1110	9	90	810	9990	899,100

Table A.21.6-1 : Cost estimate for groyne type A

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST		
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk	
1	2	3	4	5	6	7	8	9	10	
8	Supplying & sinking steel piles dia 50cm, t = 10mm	P4	m ²	8060	26	120	3120	209560	25,147,200	
9	Bracing of steel or C.C. piles	P9	m	5000	5	120	600	25000	3,000,000	
10	Production of C.C. blocks (1:3:6) with brick khola & coarse sand	G11	m ³	2800	40	65	2600	112000	7,280,000	
11	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000	
							Total	411994	42,738,940	

Table A.21.6-1 (continued) : Cost estimate for groyne type A

DESCRIPTION:



On the flood plain : a) concrete sheet pile coffer dam

b) bed protection upstream $l = 30m$
downstream $l = 30m$, boulders

In the river : a) steel pile structure two rows dia 50cm, $t = 10mm$
permeability 50%

b) bed protection on both sides, sand bags

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Supplying & sinking C.C. sheet piles (6x0.75x0.2)	P8	m ²	3000	6	128	768	18000	2,304,000
2	Bracing of steel or C.C. piles	P9	m	5000	1	128	128	5000	640,000
3	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	20	60	1200	700	42,000
4	Compaction with vibration plate	G7	m ³	10	20	60	1200	200	12,000
5	Supply, spreading & levelling of coarse sand $t = 0.15m$	AF1	m ²	140	8	60	480	1120	67,200
6	Herring bone bond bricks in 2 layer encased with two layer wire mesh	AP1	m ²	403	8	60	480	3224	193,440
7	Supplying & placing above water by 40-60kg. boulder ($t = 0.5m$) manually	AP4	m ³	1850	4	60	240	7400	444,000

Table A.21.6-2 : Cost estimate for groyne type B

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST		
					Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total	Total Tk
1	2	3	4	5	6	7	8	9	10	
8	Supplying & sinking steel piles dia 50cm, t = 10mm	P4	m ²	8060	26	120	3120	209560	25,147,200	
9	Bracing of steel or C.C. piles	P9	m	5000	5	120	600	25000	3,000,000	
10	Supply & placing of geosynthetic sand bags (0.5 m ³) 1.2x1.3 = 1.5 m ² , Type 5	G10	m ³	1570	40	65	2600	62800	4,082,000	
11	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000	
							Total	338204	36,269,840	

Table A.21.6-2 (continued) : Cost estimate for groyne type B

40.0m
25.0m
15.0m
20.0m
80.0m
60.0m
8.0m

SERVICE AND MONITORING ROADWAY

A-A
B-B
C-C

STEEL PILES OF 50x10 CM

EARTH DAM

1:100

1:250

b) bed protection both sides \ l=30m, boulders

In the river

: a) steel pile structure two rows dia 50cm, d=10mm

b) bed protection : both sides chemical

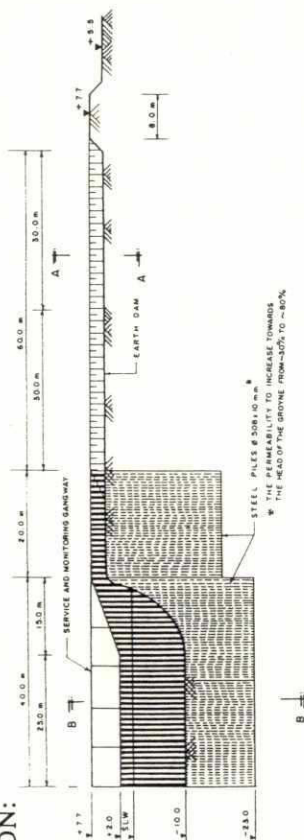
bonded blocks

c) permeability increasing 30% to 80%

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	32.5	60	1950	1137.5	68,250
2	Compaction with vibration plate	G7	m ³	10	32.5	60	1950	325	19,500
3	Supply, spreading & levelling of coarse sand t = 0.15m	AF1	m ²	140	9	60	540	1260	75,600
4	Supplying, spreading & levelling of down graded khoa t = 0.15m	AF3	m ²	141	9	60	540	1269	76,140
5	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	AP4	m ²	925	9	60	540	8325	499,500
6	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	AP4	m ³	1850	4	60	240	7400	444,000
7	Supplying & sinking steel piles dia 50cm, t = 10mm	P4	m ²	8060	26	120	3120	209560	25,147,200
8	Bracing of steel or C.C. piles	P9	m	5000	5	120	600	25000	3,000,000
9	Production of chemical bonded blocks	G14	m ³	1665	40	65	2600	66600	4,329,000
10	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	326076.5	33,997,190

Table A.21.6-3 : Cost estimate for groyne type C

DESCRIPTION:



On the flood plain : a) earth dam, protected

b) no bed protection

In the river

: a) steel pile structure two rows dia 50cm

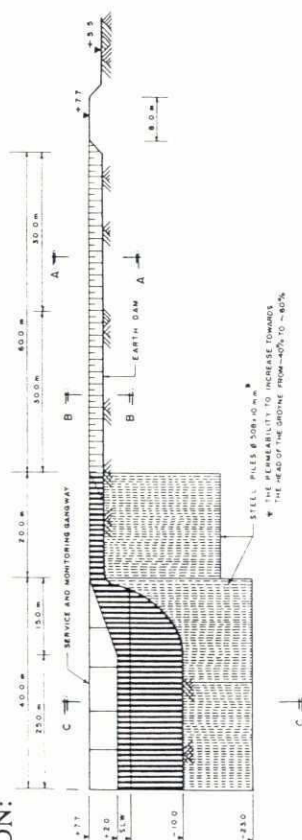
t = 10mm permeability increasing 30% to 80%

b) no bed protection

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	32.5	60	1950	1137.5	68,250
2	Compaction with vibration plate	G7	m ³	10	32.5	60	1950	325	19,500
3	Supplying and placing of sand asphalt t = 0.15m	AF2	m ²	326	18	60	1080	5868	352,080
4	Supplying & placing above water by 40-60kg. boulder t = 0.5m with bituminous grout 35 l/m ² (70:16:14) manually	AP6	m ²	1215	9	60	540	10935	656,100
5	Supplying & sinking steel piles dia 50cm, t = 10mm	P4	m ²	8060	26	120	3120	209560	25,147,200
6	Bracing of steel or C.C. piles	P9	m	5000	5	60	300	25000	1,500,000
							Total	252825.5	27,743,130

Table A.21.6-4 : Cost estimate for groynes type D

DESCRIPTION:



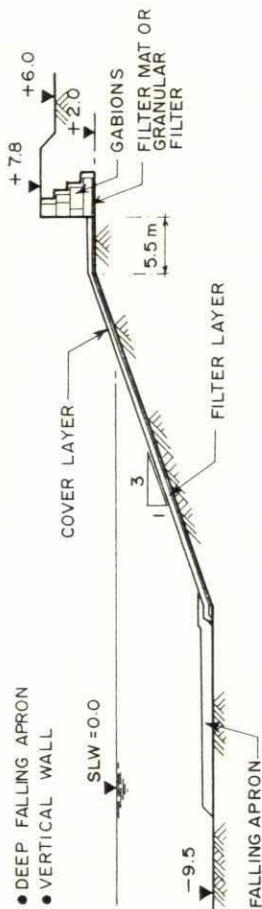
- On the flood plain : a) earth dam, protected
b) bed protection both sides $l=30m$,
chemical bonded blocks
- In the river : a) steel pile structure two rows dia 50cm, $d=10mm$
b) bed protection : both sides, sand bags
c) permeability increasing 40% to 60%

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	32.5	60	1950	1137.5	68,250
2	Compaction with vibration plate	G7	m ³	10	32.5	60	1950	325	19,500
3	Supply, spreading & levelling of coarse sand $t = 0.15m$	AF1	m ²	140	9	60	540	1260	75,600
4	Supplying, spreading & levelling of down graded khoa $t = 0.15m$	AF3	m ²	141	9	60	540	1269	76,140
5	Production of chemical bonded blocks	G14	m ³	1665	4.5	60	270	7492.5	449,550
6	Production of chemical bonded blocks	G14	m ³	1665	4	60	240	6660	399,600
7	Supplying & sinking steel piles dia 50cm, $t = 10mm$	P4	m ²	8060	26	120	3120	209560	25,147,200
8	Bracing of steel or C.C. piles	P9	m	5000	5	120	600	25000	3,000,000
9	Supply & placing of geosynthetic sand bags (0.5 m ³) $1.2 \times 1.3 = 1.5 m^2$, Type 5	G10	m ³	1570	40	65	2600	62800	4,082,000
10	Dumping of boulders C.C. blocks, gabions etc. from barges	BP4	m ³	130	40	65	2600	5200	338,000
							Total	320704	33,655,840

Table A.21.6-5 : Cost estimate for groyne type E

DESCRIPTION:

TYPE BA - 6

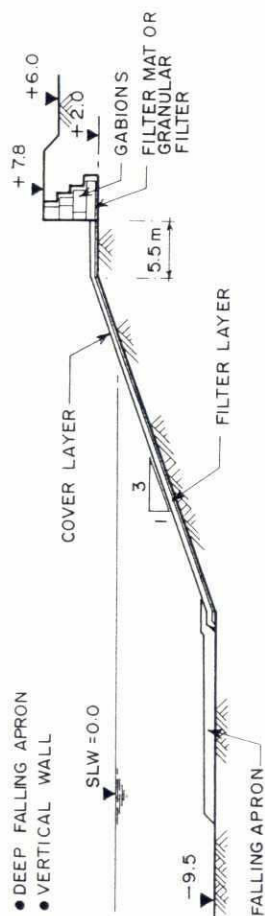


FILTER A W: FA4
 FILTER B W: FB3
 COVER A W: A1
 COVER B W: B1
 TOE PROTECTION: T1
 VERTICAL WALL: V3

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 2	FA4	AF4	m ²	195	28.5	100	2850	5557.5	555,750
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	34	100	3400	16830	1,683,000
3	Supplying & placing above water by 40-60kg. boulder (t = 0.5m) manually	A1	AP4	m ²	925	12	100	1200	11100	1,110,000
4	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	B1	BP1	m ³	1870	16	100	1600	29920	2,992,000
5	Supplying and dumping of boulders or rocks (10-60 kg) from barge t = 0.5m	T1	BP1	m ³	1870	24	100	2400	44880	4,488,000
6	Manufacturing & placing gabions with steel wire mesh 1x1x0.5m filled with boulders placed above water	V3	AP10	m ³	2632	14	100	1400	36848	3,684,800
								Total	145135.5	14,513,550

Table A.21.6-6: Cost estimate of embankment test structures No A

TYPE BA - 6



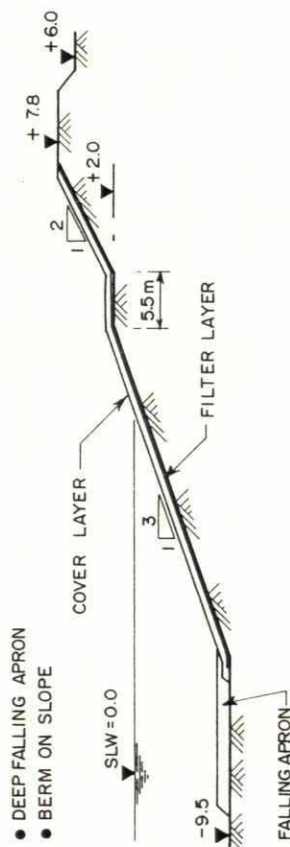
FILTER A W : FA4
 FILTER B W : FB3
 COVER A W : A2
 COVER B W : B2
 TOE PROTECTION : T3
 VERTICAL WALL : V2

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/m)	Total
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 2	FA4	AF4	m ²	195	28.5	100	2850	5557.5	555,750
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	34	100	3400	16830	1,683,000
3	Production of chemical bonded blocks	A2	G14	m ³	1665	6	100	600	9990	999,000
4	Placing of C.C. blocks above water by crane (various sizes)	A2	AP8	m ²	120	12	100	1200	1440	144,000
5	Production of chemical bonded blocks	B2	G14	m ³	1665	16	100	1600	26640	2,664,000
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B2	BP2	m ²	500	32	100	3200	16000	1,600,000
7	Supply & dumping of geotextile sand bags (G10) + (BP4)	T3	BP7	m ³	1700	20	100	2000	34000	3,400,000
8	Manufacturing & placing gabions with steel wire mesh 1x1x0.5m filled with bricks placed above water	V2	AP9	m ³	1632	14	100	1400	22848	2,284,800
								Total	133305.5	13 330 550

Table A. 21.6-7 : Cost estimate of embankment test structures No B

TYPE BA - 5

DESCRIPTION:



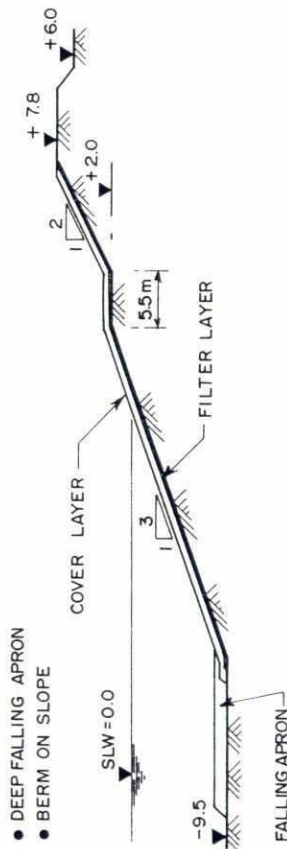
FILTER A W : FA4
 FILTER B W : FB3
 COVER A W : A6
 COVER B W : B5
 TOE PROTECTION : T2

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	1 Supply, spreading & levelling of coarse sand t = 0.15m	FA1	AF1	m ²	140	25	100	2500	3500	350,000
2	2 Supplying, spreading & levelling of down graded khoa t = 0.15m	FA1	AF3	m ²	141	25	100	2500	3525	352,500
3	3 Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	45.5	100	4550	13650	1,365,000
4	4 Supplying & placing above water by 40-60kg. boulder t = 0.5m with bituminous grout 35 l/m ² (70:16:14) manually	A4	AP6	m ²	1215	18	100	1800	21870	2,187,000
5	5 Supply & dumping of geotextile sand bags (G10) + (BP4)	B9	BP7	m ³	1700	38	100	3800	64600	6,460,000
8	8 Supply & dumping of geotextile sand bags (G10) + (BP4)	T3	BP7	m ³	1700	13	100	1300	22100	2,210,000
								Total	129245	12,924,500

Table A.21.6-8 : Cost estimate of embankment test structures No C

TYPE BA - 5

DESCRIPTION:



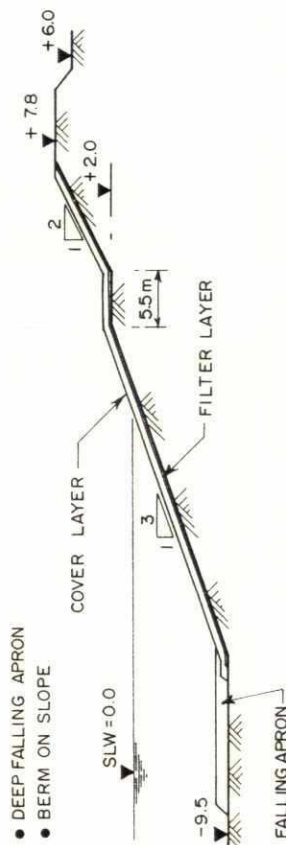
FILTER A W : FA4
 FILTER B W : FB3
 COVER A W : A6
 COVER B W : B5
 TOE PROTECTION : T2

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 2	FA4	AF4	m ²	195	12.5	100	1250	2437.5	243,750
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	45.5	100	4550	22522.5	2,252,250
3	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	A6	G11	m ³	2800	7.2	100	720	20160	2,016,000
4	Placing of C.C. blocks above water by crane (various sizes)	A6	AP8	m ²	120	18	100	1800	2160	216,000
5	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	B5	G11	m ³	2800	15.2	100	1520	42560	4,256,000
6	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B5	BP2	m ²	500	38	100	3800	19000	1,900,000
7	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	T2	G11	m ³	2800	13	100	1300	36400	3,640,000
8	Dumping of boulders C.C. blocks, gabions etc. from barges	T2	BP4	m ³	130	13	100	1300	1690	169,000
								Total	146930	14,693,000

Table A. 21.6-9 : Cost estimate of embankment test structures No D

TYPE BA - 5

DESCRIPTION:



FILTER A W : FA2
 FILTER B W : FB2
 COVER A W : A3
 COVER B W : B5
 TOE PROTECTION : T7

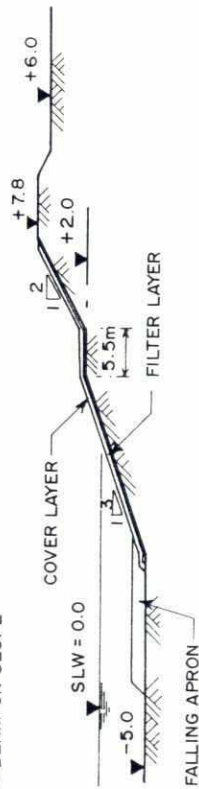
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of sand asphalt t = 0.15m	FA2	AF2	m ²	326	12.5	100	1250	4075	407,500
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 2	FB2	BF2	m ²	346	45.5	100	4550	15743	1,574,300
3	Supplying and placing boulders above water t = 0.5m with cement grout 50 l/m ²	A3	AP12	m ²	1085	18	100	1800	19530	1,953,000
4	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	B5	G11	m ³	2800	15.2	100	1520	42560	4,256,000
5	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B5	BP2	m ²	500	38	100	3800	19000	1,900,000
6	Production of C.C. blocks (1:3:6) with brick khoa & coarse sand	T2	G11	m ³	2800	19.2	100	1920	53760	5,376,000
7	Dumping of boulders C.C. blocks, gabions etc. from barges	T2	BP4	m ³	130	19.2	100	1920	2496	249,600
								Total	157164	15,716,400

Table A.21.6-10 : Cost estimate of embankment test structures No E

DESCRIPTION:

TYPE BA -2

- SHALLOW FALLING APRON
- BERM ON SLOPE



FILTER A W : FA2
 FILTER B W : FB1
 COVER A W : A8
 COVER B W : B10
 TOE PROTECTION : T5

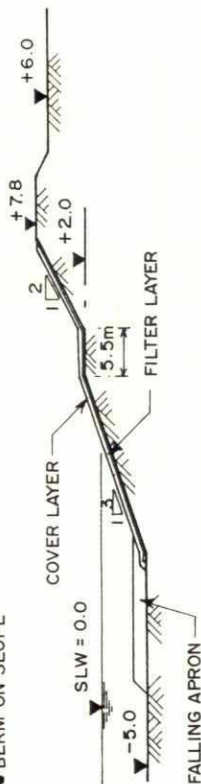
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of sand asphalt t = 0.15m	FA2	AF2	m ²	326	12.5	100	1250	4075	407,500
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	29.5	100	2950	8850	885,000
3	Open stone asphalt above water t = 0.15m	A8	AP2	m ²	820	12.5	100	1250	10250	1,025,000
4	Tubular fabrics filled with sand	B10	BP8	m ²	680	28	100	2800	19040	1,904,000
5	Tubular fabrics filled with sand	T5	BP8	m ²	680	15	100	1500	10200	1,020,000
								Total	52415	5,241,500

Table A.26.6-11 : Cost estimate of embankment test structures No F

DESCRIPTION:

TYPE BA -2

- SHALLOW FALLING APRON
- BERM ON SLOPE

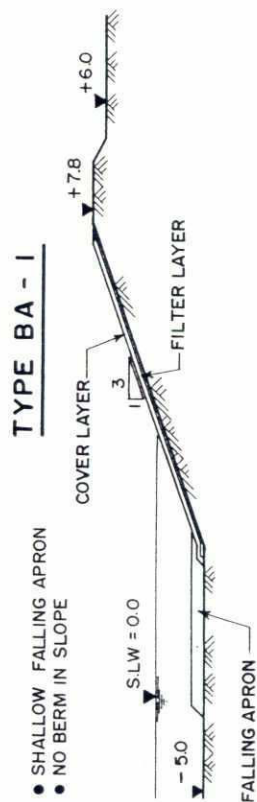


FILTER A W : FA3
 FILTER B W : FB2
 COVER A W : A11
 COVER B W : B12
 TOE PROTECTION : T6

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	12.5	100	1250	1875	187,500
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	29.5	100	2950	14602.5	1,460,250
3	Manufacturing & placing gabions with steel wire mesh 1x1x0.5m filled with boulders placed above water	A11	AP10	m ²	1316	12.5	100	1250	16450	1,645,000
4	Manufacturing of gabions with boulders (various sizes)	B12	AP11	m ³	2392	14	100	1400	33488	3,348,800
5	Placing of C.C. blocks below water by crane, barge, tug boat etc. block size 0.5x0.5x0.5m	B12	BP2	m ²	500	28	100	2800	14000	1,400,000
6	Manufacturing of gabions with boulders (various sizes)	T6	AP11	m ³	2392	10.8	100	1080	25833.6	2,583,360
7	Dumping of boulders C.C. blocks, gabions etc. from barges	T6	BP4	m ³	130	10.8	100	1080	1404	140,400
								Total	107653.1	10,765,310

Table A.21.6-12 : Cost estimate of embankment test structures No G

DESCRIPTION:

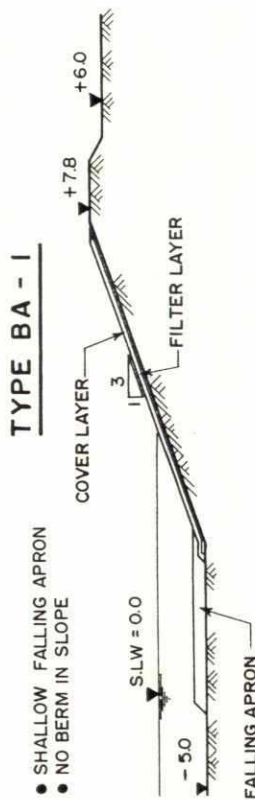


FILTER A W : FA4
FILTER B W : FB3
COVER A W : B6
COVER B W : B6
TOE PROTECTION : T4

ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES		COST	
						Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)
1	2	3	4	5	6	7	8	9	10
1	Supplying and placing of geosynthetic filter mat above water Type 2	FA4	AF4	m ²	195	21	100	2100	4095
2	Supply & placing of geosynthetic sand mat Type 3	FB3	BF3	m ²	495	21	100	2100	10395
3	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m A.W.	B6	BP5	m ²	1475	21	100	2100	30975
4	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m B.W.	B6	BP5	m ²	1475	19	100	1900	28025
5	Supply, manufacture and placing of cable connected C.C. blocks t = 0.25m	T4	BP5	m ²	1475	10	100	1000	14750
								Total	88240
								Total	8,824,000

Table A.21.6-13 : Cost estimate of embankment test structures No H

DESCRIPTION:



FILTER A W : FA3
FILTER B W : FB1
COVER A W : B7
COVER B W : B7
TOE PROTECTION : T7

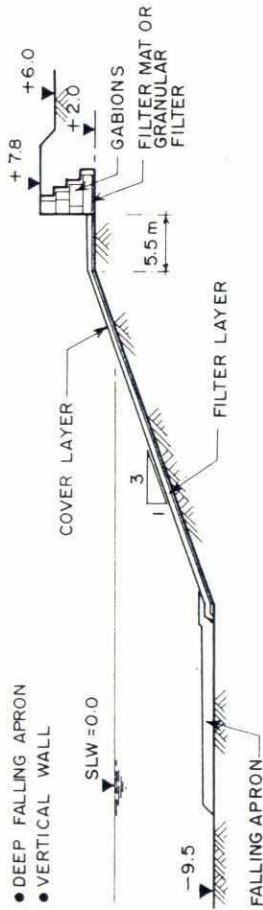
ITEM NOS	DESCRIPTION OF ITEM	WORK NO	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
						Specific (Units/m)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10	11
1	Supplying and placing of geosynthetic filter mat above water Type 1	FA3	AF5	m ²	150	21	100	2100	3150	315,000
2	Supplying and placing of geosynthetic filter mat below water with brick ballast sinking Type 1	FB1	BF1	m ²	300	19	100	1900	5700	570,000
3	Supply, manufacture and placing of C.C. blocks connected to geotextile t = 0.35m Type 2 A.W.	B7	BP6	m ²	1774	21	100	2100	37254	3,725,400
4	Supply, manufacture and placing of C.C. blocks connected to geotextile t = 0.35m Type 2 B.W.	B7	BP6	m ²	1774	19	100	1900	33706	3,370,600
5	Supply and placing of geotextile scour prevention mat Type 4	T7	BP3	m ²	1300	16	100	1600	20800	2,080,000
								Total	100610	10,061,000

Table A.21.6-14 : Cost estimate of embankment test structure No K

319

DESCRIPTION:

TYPE BA - 6



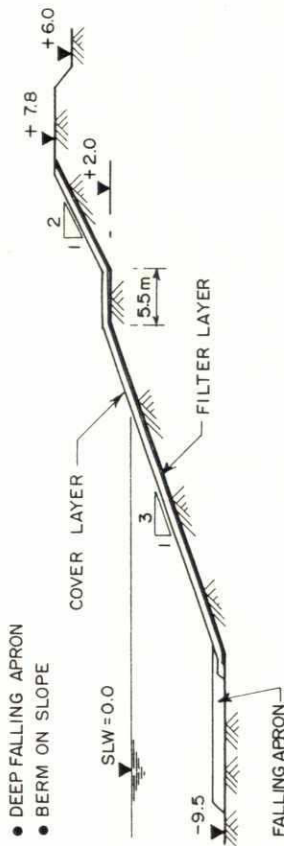
EARTH WORKS

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Hydraulic excavation by dredger 0 - 3000m 3000m - 4000m 4000m - 5000m	G4	m ³	65	935	100	93500	60775	6,077,500
2	Excavation by Dragline/Excavator	G3	m ³	67	84	100	8400	5628	562,800
3	Excavation by Dragline/Excavator	G3	m ³	67	32	100	3200	2144	214,400
4	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	30	100	3000	1050	105,000
5	Compaction with vibration plate	G7	m ³	10	30	100	3000	300	30,000
6	Slope preparation above water by excavator	G16	m ²	40	6.5	100	650	260	26,000
7	Slope preparation below water with dragline	G15	m ²	77	32	100	3200	2464	246,400
							Total	72621	7,262,100

Table A.21.6-15 : Cost estimate of embankment test structures No A-B, earth works only

TYPE BA - 5

DESCRIPTION:



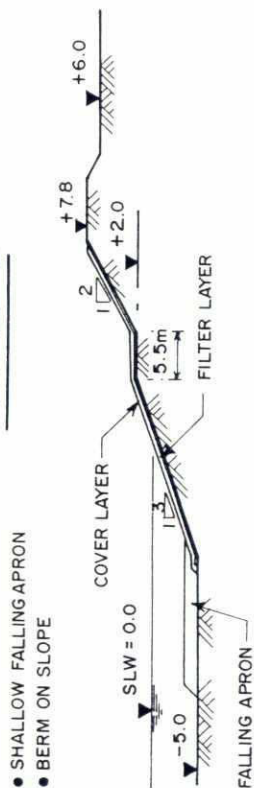
EARTH WORKS

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Hydraulic excavation by dredger 0 - 3000m 3000m - 4000m 4000m - 5000m	G4	m ³	65	935	100	93500	60775	6,077,500
2	Excavation by Dragline/Excavator	G3	m ³	67	76	100	7600	5092	509,200
3	Excavation by Dragline/Excavator	G3	m ³	67	95	100	9500	6365	636,500
4	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	30	100	3000	1050	105,000
5	Compaction with vibration plate	G7	m ³	10	30	100	3000	300	30,000
6	Slope preparation above water by excavator	G16	m ²	40	12.5	100	1250	500	50,000
7	Slope preparation below water with dragline	G15	m ²	77	38	100	3800	2926	292,600
							Total	77008	7,700,800

Table A.21.6-16 : Cost estimate of embankment test structures No C-D-E, earth works only

DESCRIPTION:

TYPE BA -2

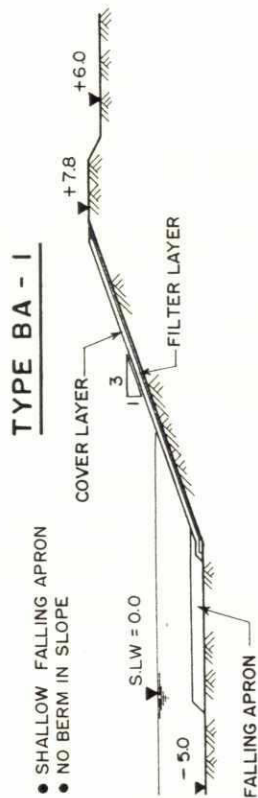


EARTH WORKS

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Hydraulic excavation by dredger 0 - 3000m 3000m - 4000m 4000m - 5000m	G4	m ³	65	560	100	56000	36400	3,640,000
2	Excavation by Dragline/Excavator	G3	m ³	67	76	100	7600	5092	509,200
3	Excavation by Dragline/Excavator	G3	m ³	67	66	100	6600	4422	442,200
4	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	30	100	3000	1050	105,000
5	Compaction with vibration plate	G7	m ³	10	30	100	3000	300	30,000
6	Slope preparation above water by excavator	G16	m ²	40	12.5	100	1250	500	50,000
7	Slope preparation below water with dragline	G15	m ²	77	27.5	100	2750	2117.5	211,750
						Total	Total	49881.5	4,988,150

Table A.21.6-17 : Cost estimate of embankment test structures No F-G, earth work only

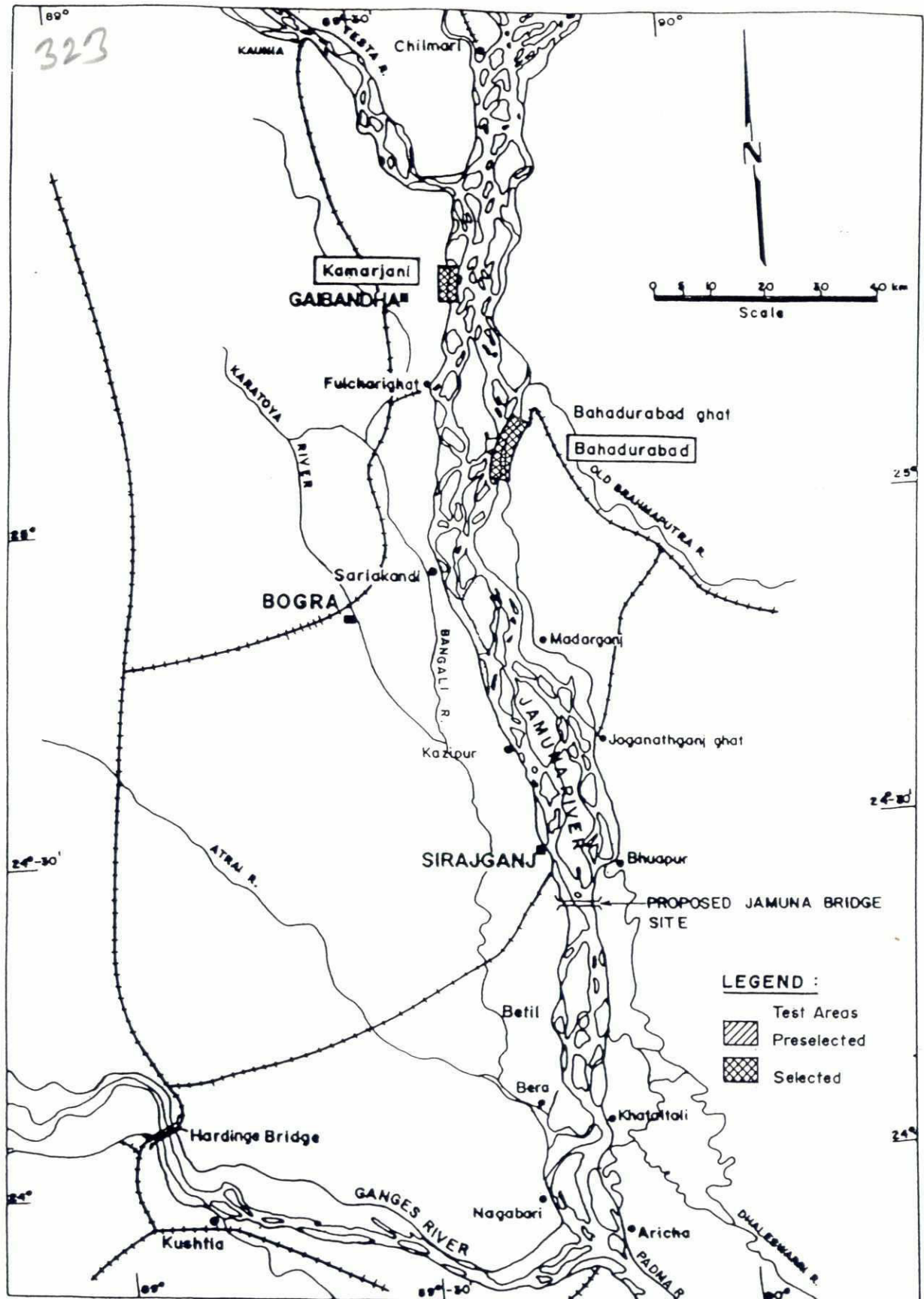
DESCRIPTION:



EARTH WORKS

ITEM NOS	DESCRIPTION OF ITEM	PRICE NO	UNIT	UNIT RATE (Tk/Unit)	QUANTITIES			COST	
					Specific (Units/lm)	linear metre (lm)	Total (Units)	Specific (Tk/lm)	Total Tk
1	2	3	4	5	6	7	8	9	10
1	Hydraulic excavation by dredger 0 - 3000m 3000m - 4000m 4000m - 5000m	G4	m ³	65	560	100	56000	36400	3,640,000
2	Excavation by Dragline/Excavator	G3	m ³	67	54	100	5400	3618	361,800
3	Excavation by Dragline/Excavator	G3	m ³	67	57	100	5700	3819	381,900
4	Excavation by hand including backfill lift > 2m and lead 30m	G2	m ³	35	30	100	3000	1050	105,000
5	Compaction with vibration plate	G7	m ³	10	30	100	3000	300	30,000
6	Slope preparation above water by excavator	G16	m ²	40	21	100	2100	840	84,000
7	Slope preparation below water with dragline	G15	m ²	77	19	100	1900	1463	146,300
							Total	47490	4,749,000

Table A.21.6-18 : Cost estimate of embankment test structures No. H-K, earth works only



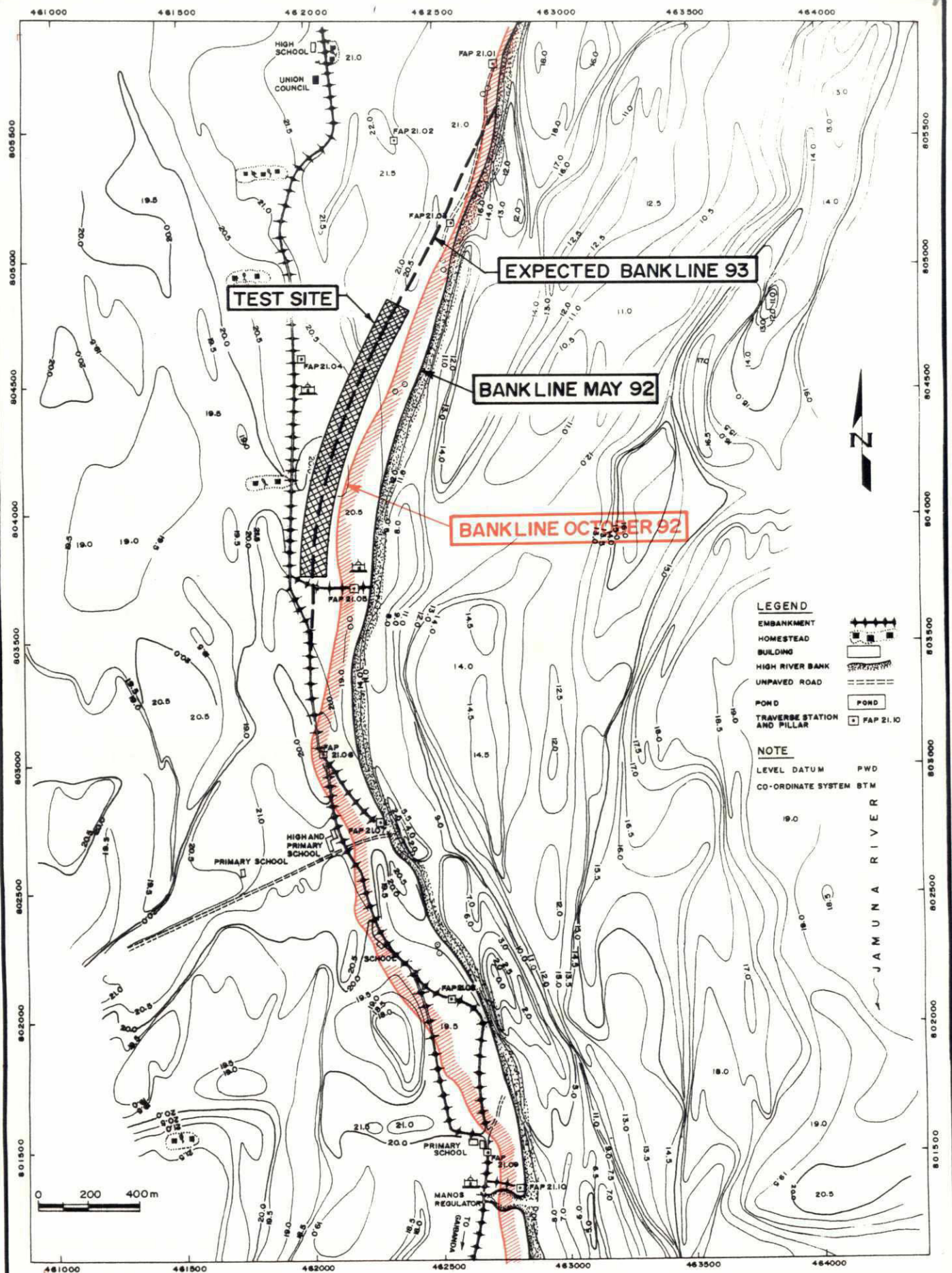
TEST AREAS

FAP - 21/22

BANK PROTECTION AND RIVER TRAINING/
(AFPM) PILOT PROJECT

FINAL REPORT

FIG. A.21 - 1

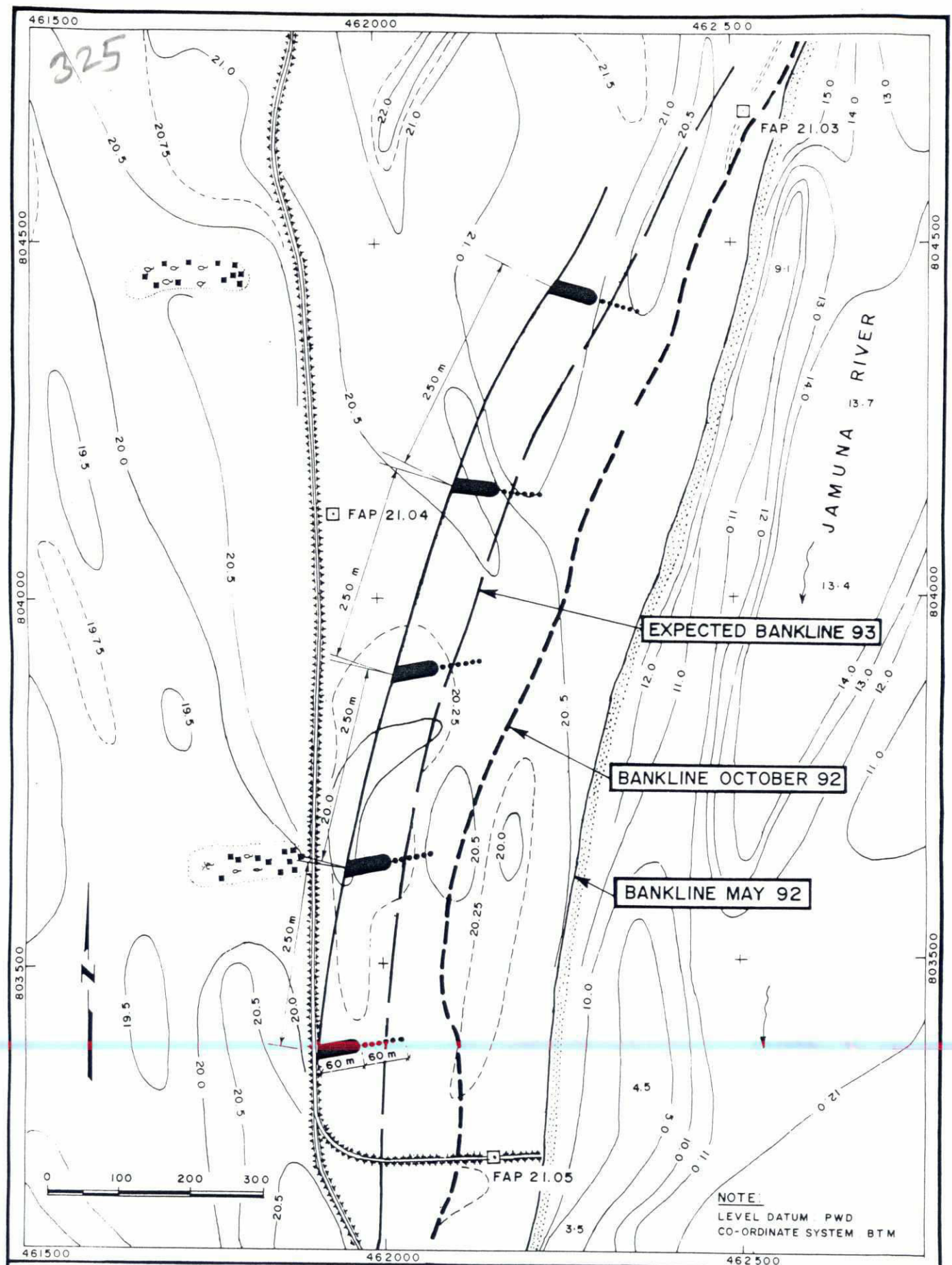


LOCATION OF TEST SITE AT KAMARJANI

FAP - 21/22
BANK PROTECTION AND RIVER TRAINING /
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FIG. A.21-2

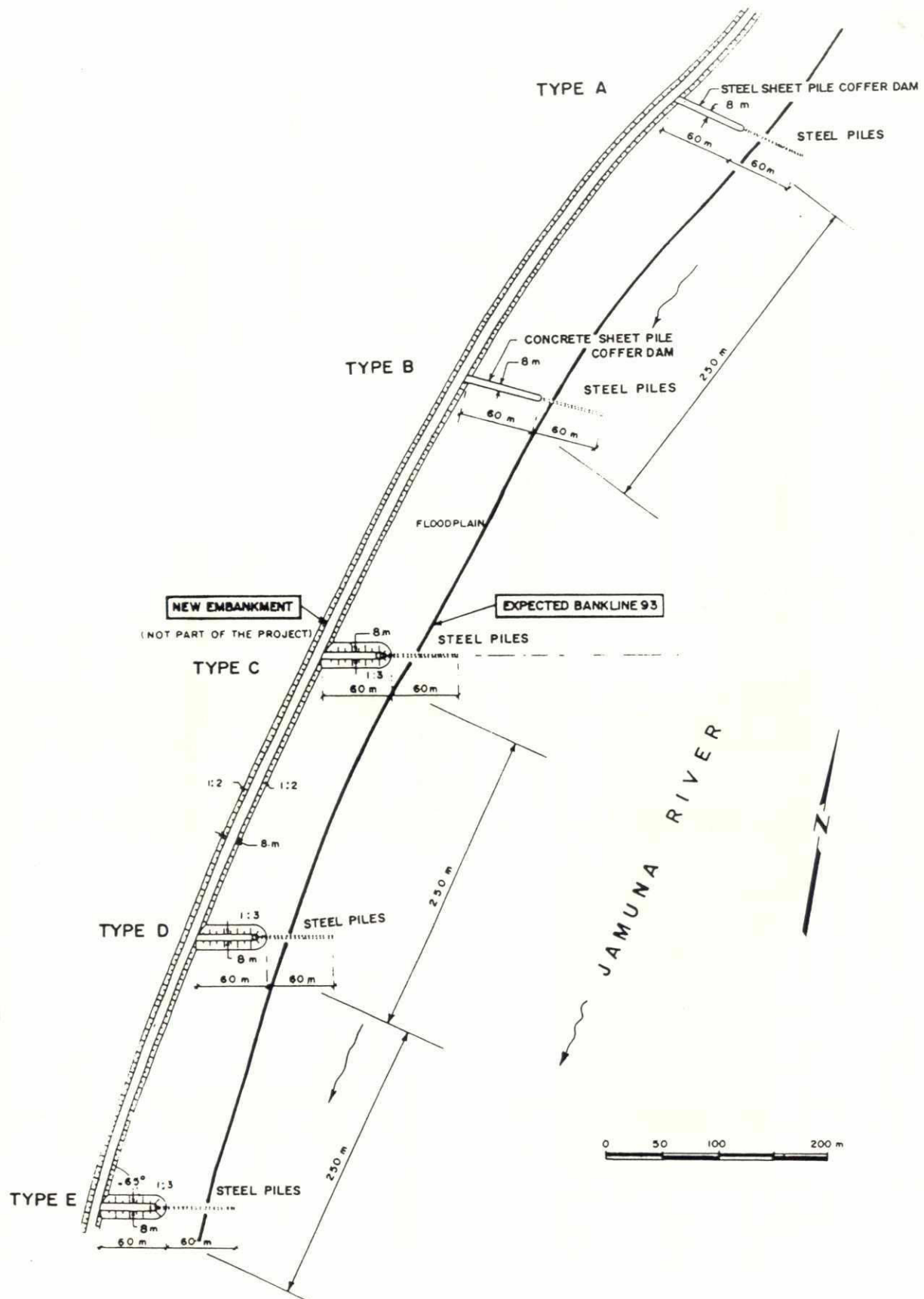


LAYOUT OF GROYNES
AT KAMARJANI TEST SITE
GENERAL LAYOUT

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FIG. A. 21 - 3



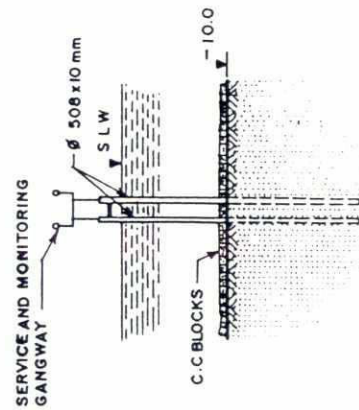
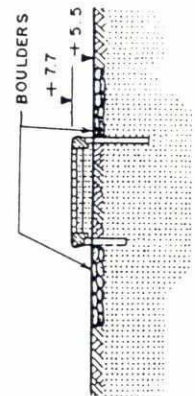
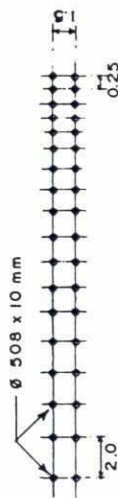
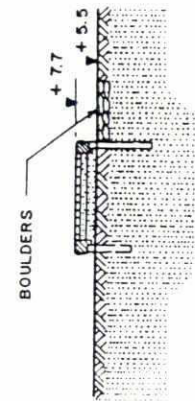
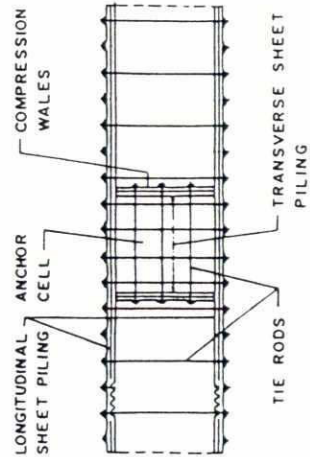
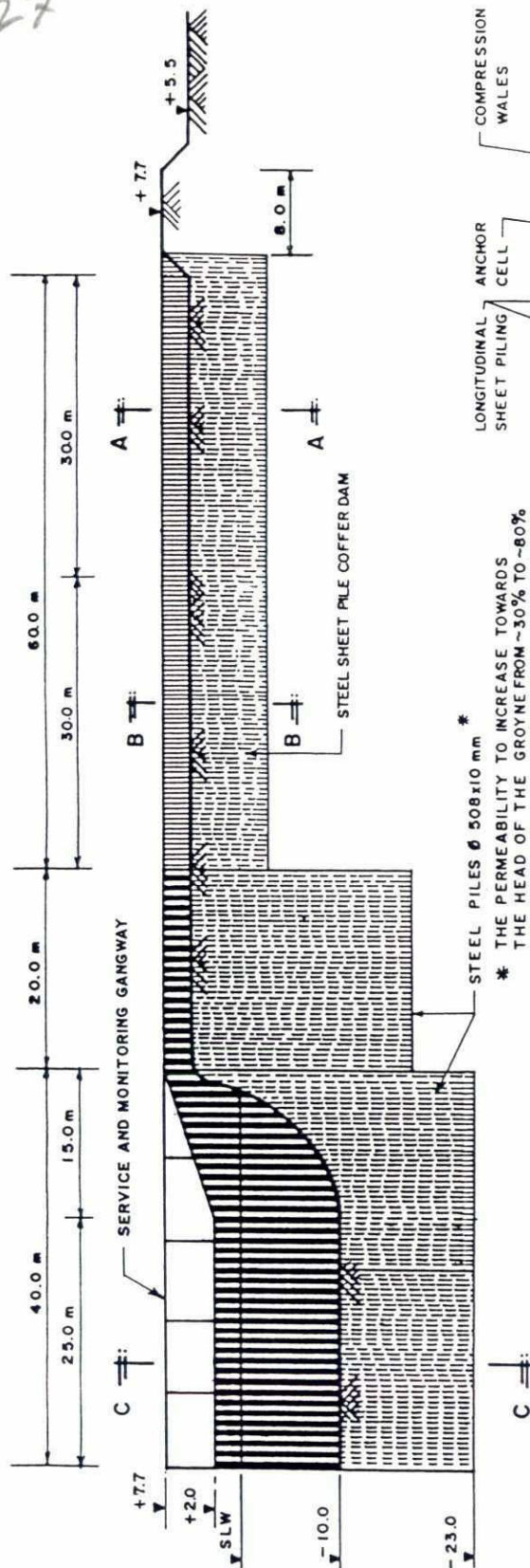
LAYOUT OF GROYNES
AT KAMARJANI TEST SITE
DETAILS

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FIG. A.21-4

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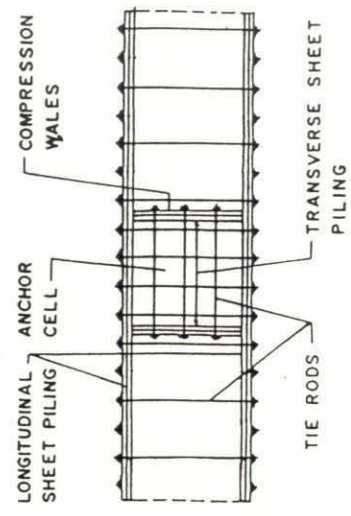
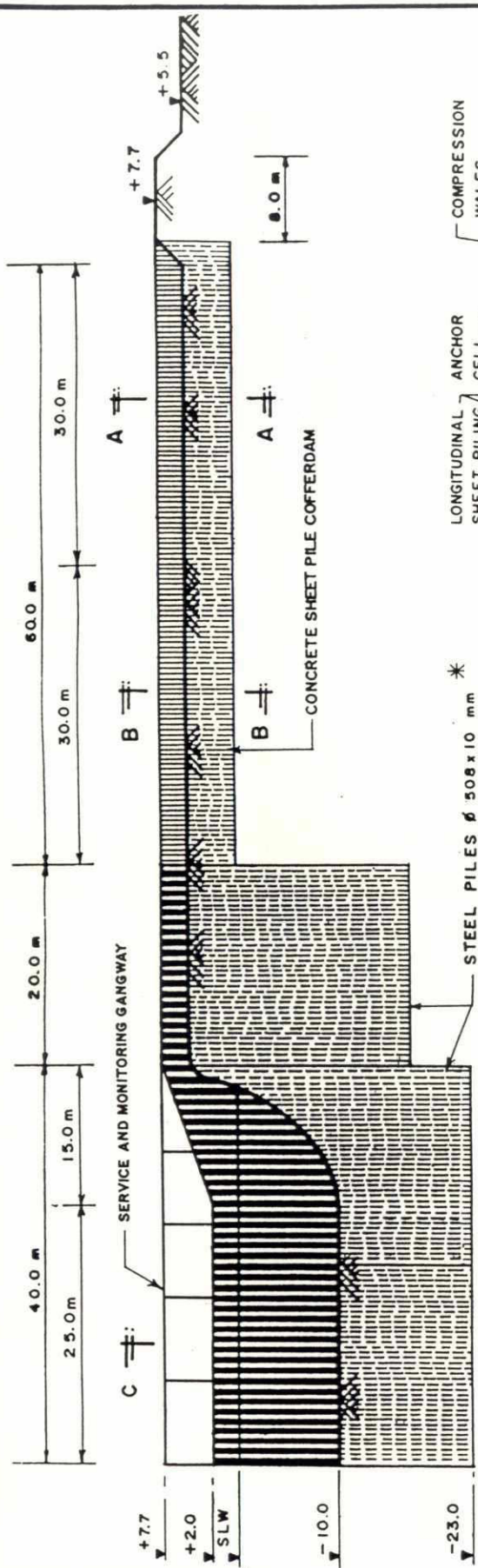
TEST STRUCTURE A
AT KAMARJANI TEST SITE

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(AFPM) PILOT PROJECT

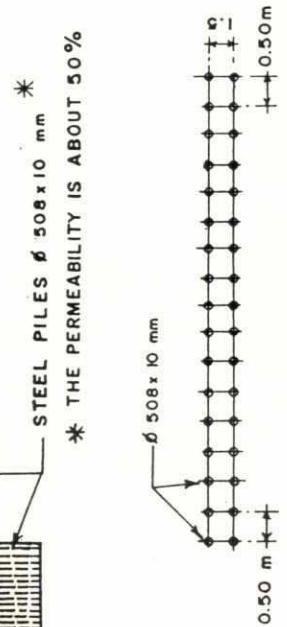
FINAL REPORT

FIG. A.21 - 5

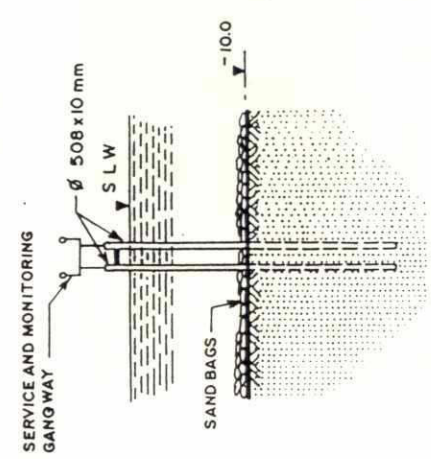
328



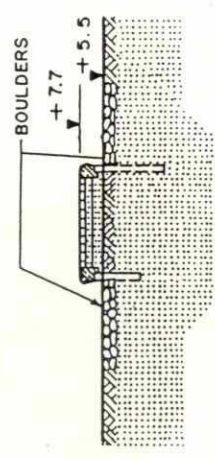
PLAN VIEW
OF SHEET PILE COFFERDAM
NOT TO SCALE



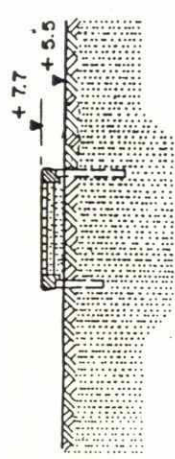
ARRANGEMENT OF PILES
(DETAIL)
NOT TO SCALE



SECTION C-C
NOT TO SCALE



SECTION B-B



SECTION A-A

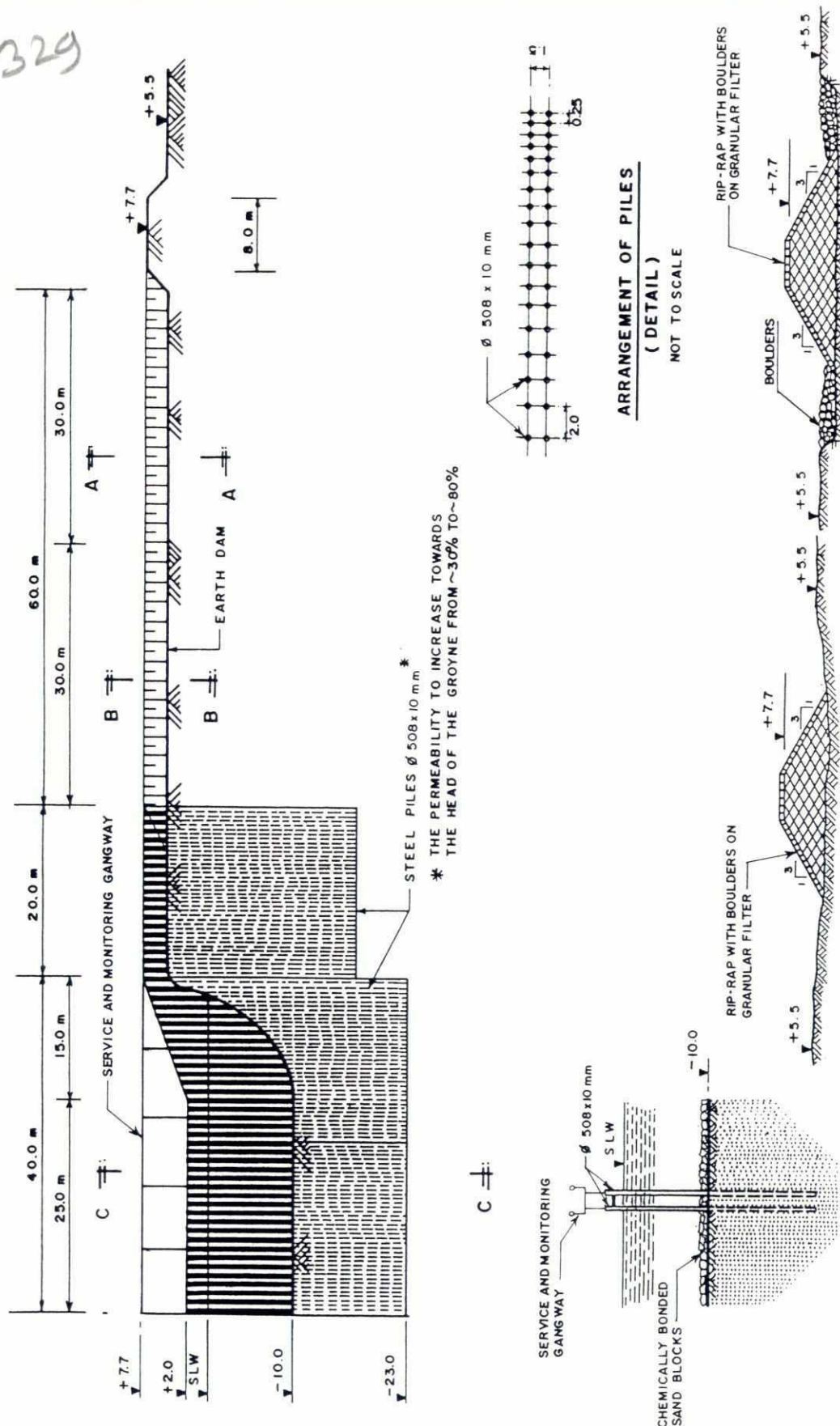


TEST STRUCTURE B
AT KAMARJANI TEST SITE

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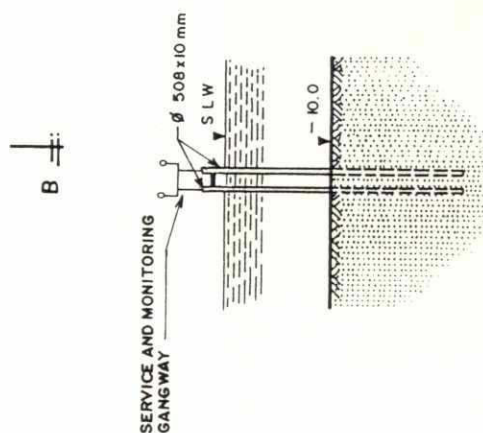
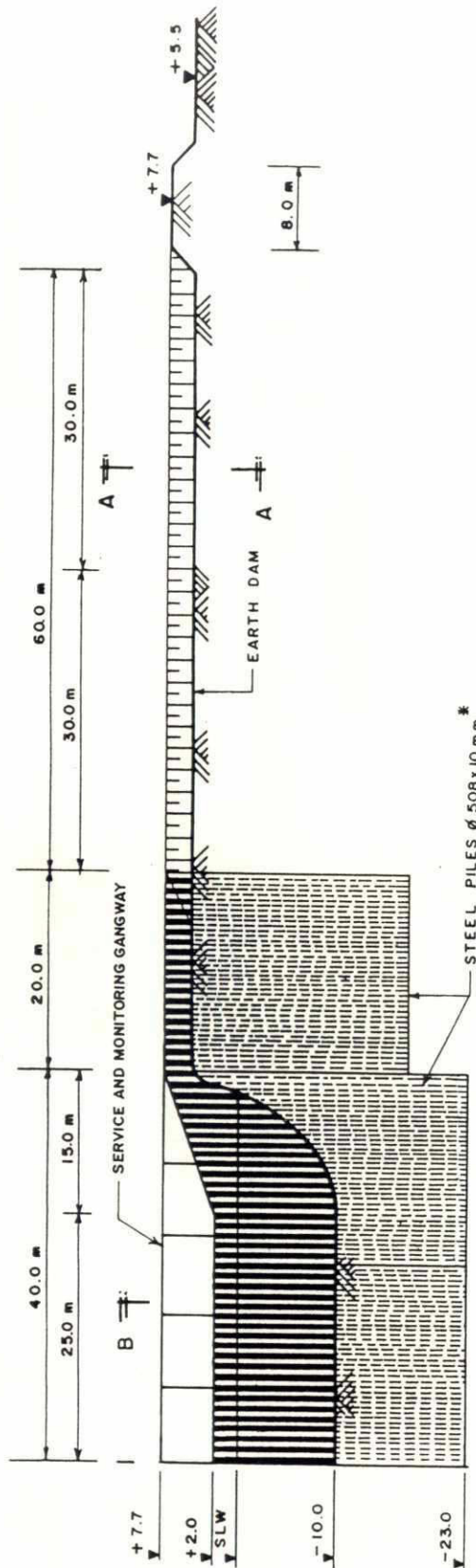
TEST STRUCTURE C AT KAMARJANI TEST SITE

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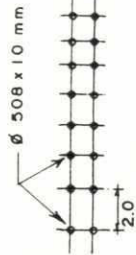
FINAL REPORT

FIG.A.21-7

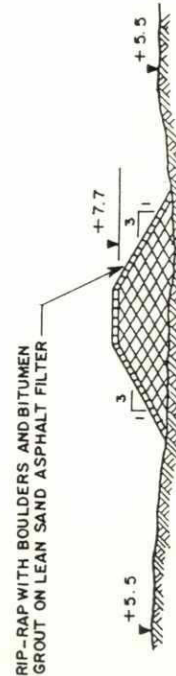
330



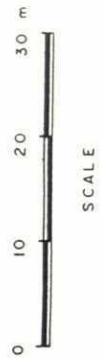
SECTION B-B
NOT TO SCALE



ARRANGEMENT OF PILES
(DETAIL)
NOT TO SCALE



SECTION A-A
NOT TO SCALE



TEST STRUCTURE D AT KAMARJANI TEST SITE

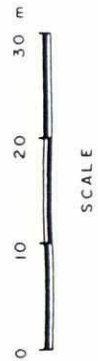
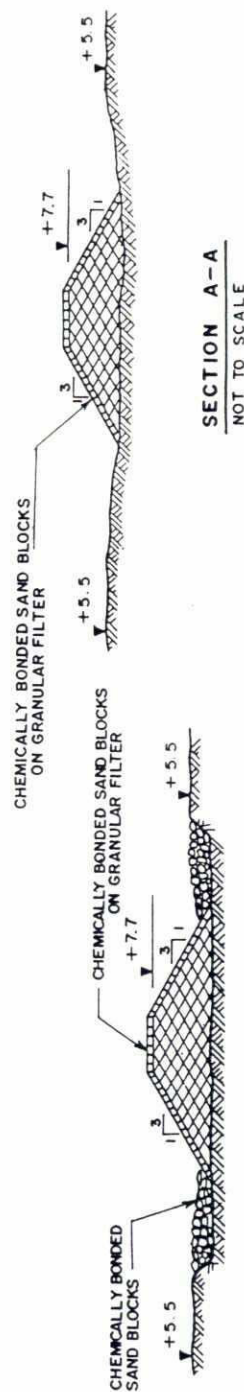
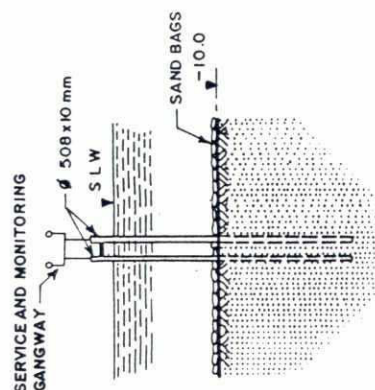
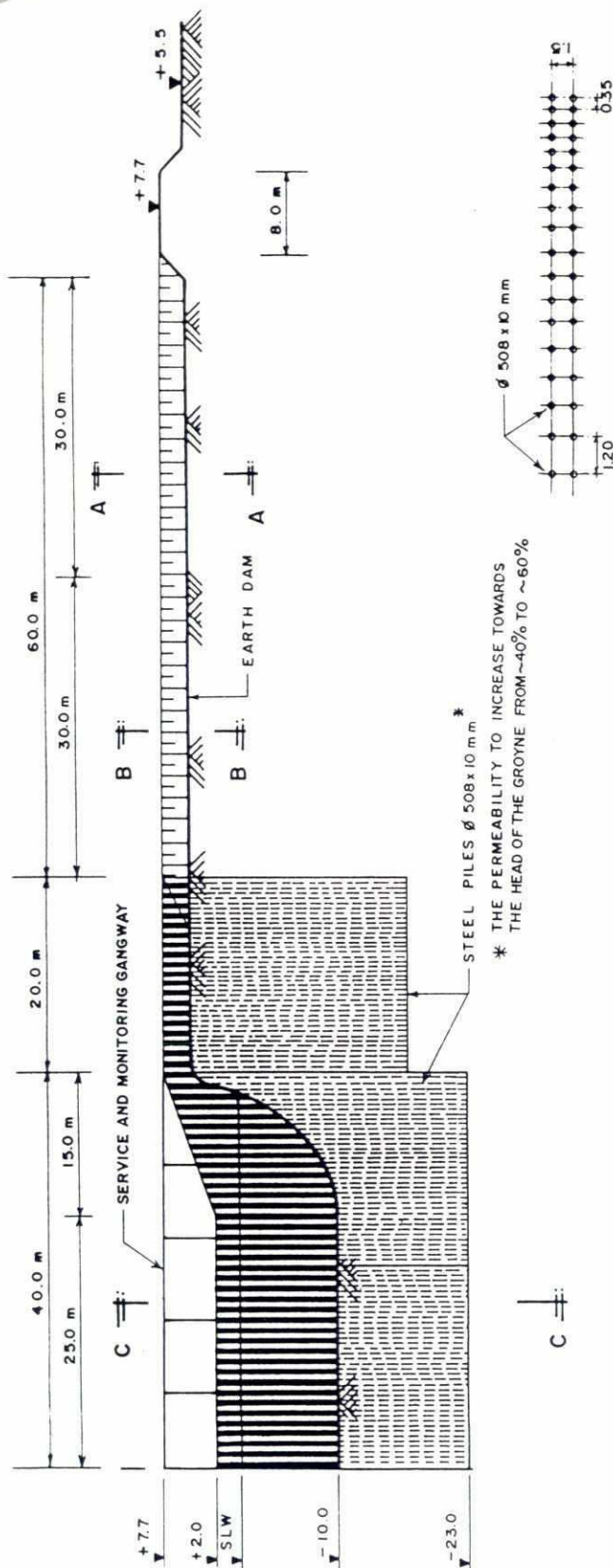
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FIG. A.21 - 8

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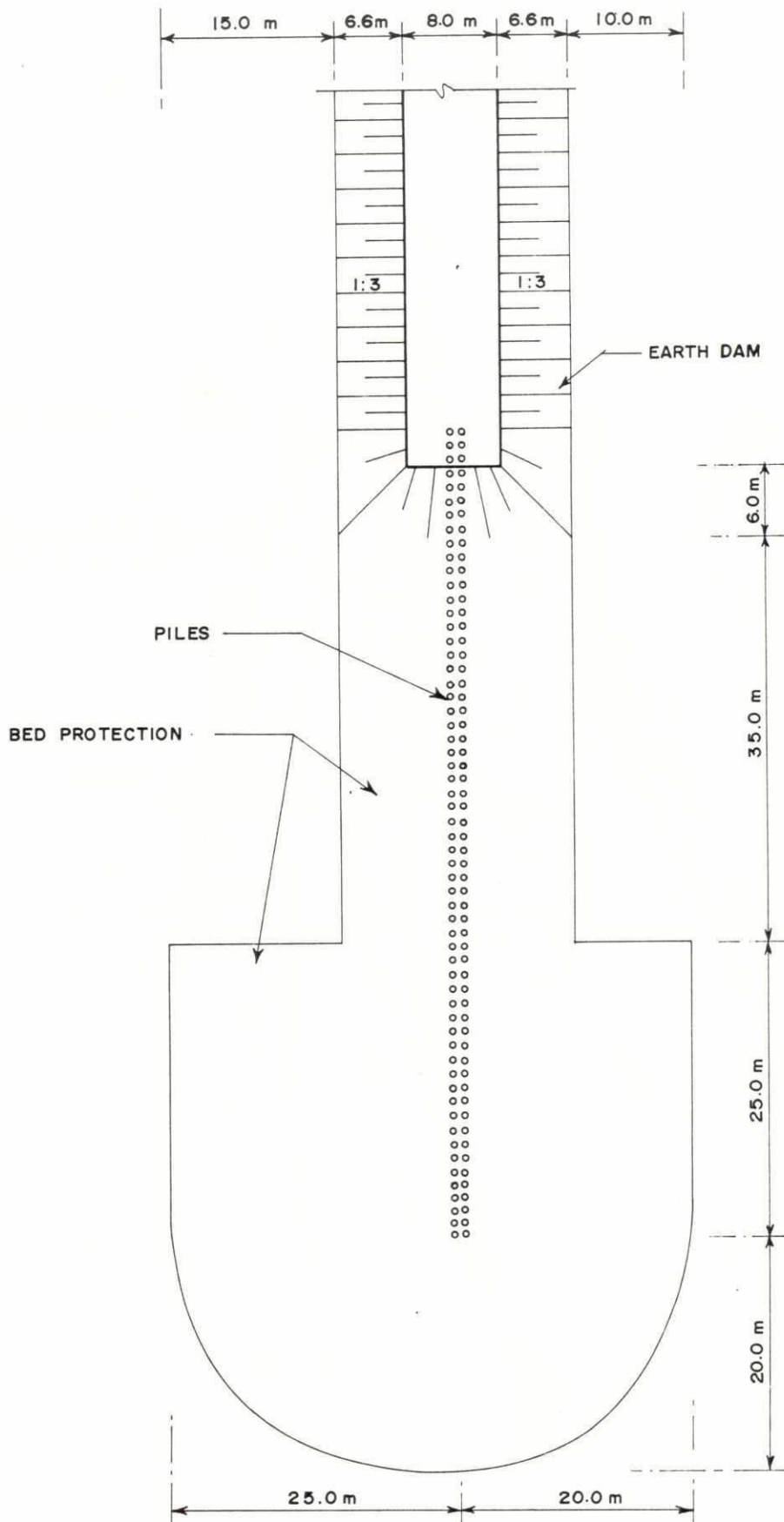
TEST STRUCTURE E AT KAMARJANI TEST SITE

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FIG. A-21-9

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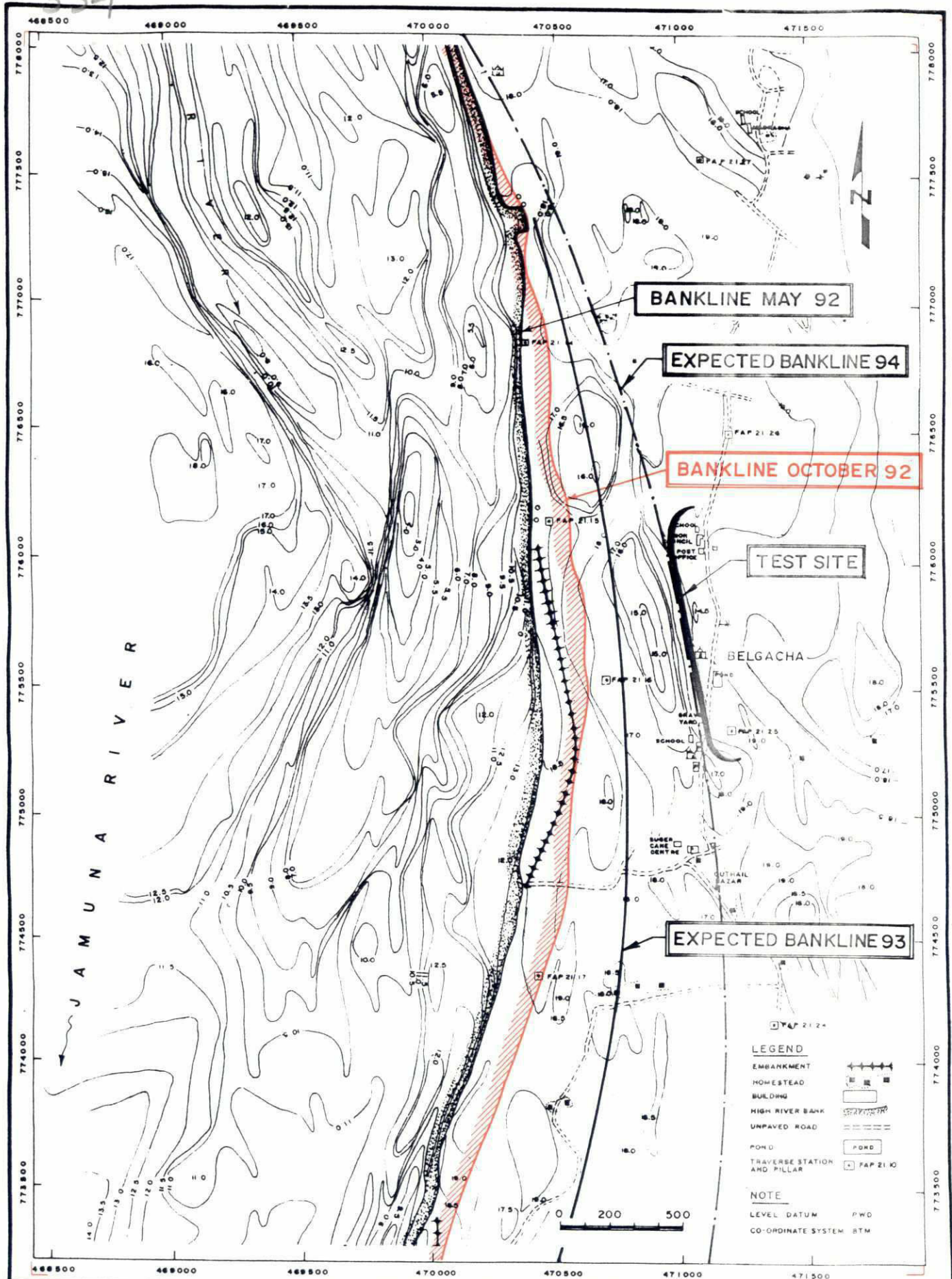


KAMARJANI TEST SITE
 DIMENSION OF BED PROTECTION
 IN THE AREA OF PILE STRUCTURE

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 (AFPM) PILOT PROJECT

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FIG.A.21-10



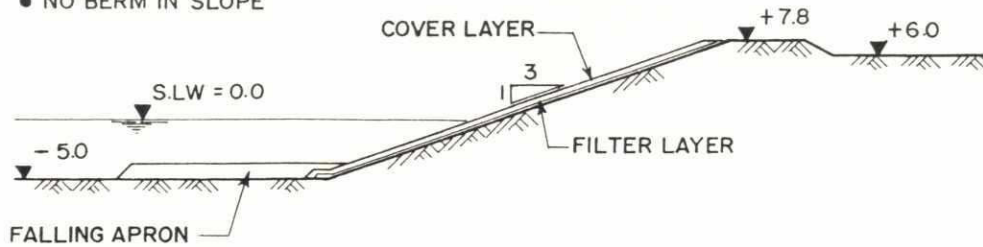
LOCATION OF TEST SITE AT BAHADURABAD

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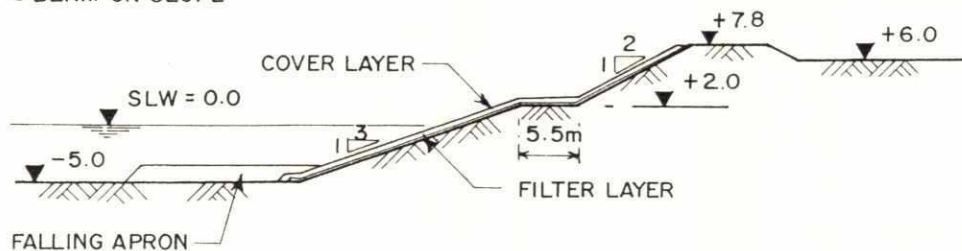
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FIG A.21 - 11

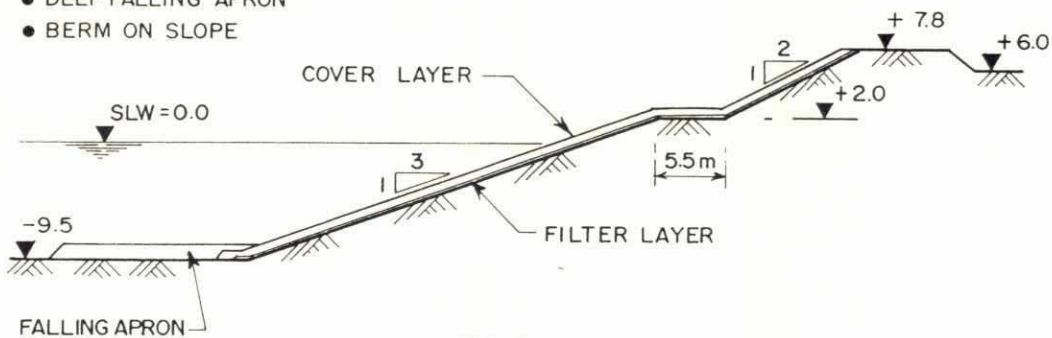
- SHALLOW FALLING APRON
- NO BERM IN SLOPE

TYPE BA - 1

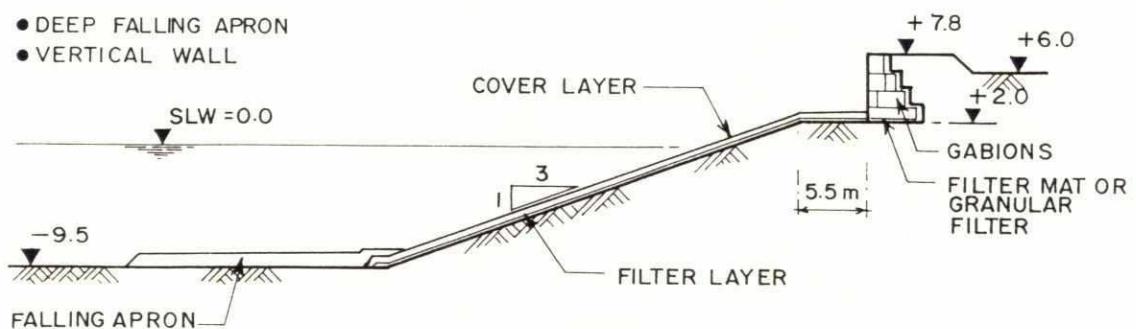
- SHALLOW FALLING APRON
- BERM ON SLOPE

TYPE BA - 2

- DEEP FALLING APRON
- BERM ON SLOPE

TYPE BA - 5

- DEEP FALLING APRON
- VERTICAL WALL

TYPE BA - 6

**CROSS SECTION OF EMBANKMENT
AT BAHADURABAD
TEST SITE**

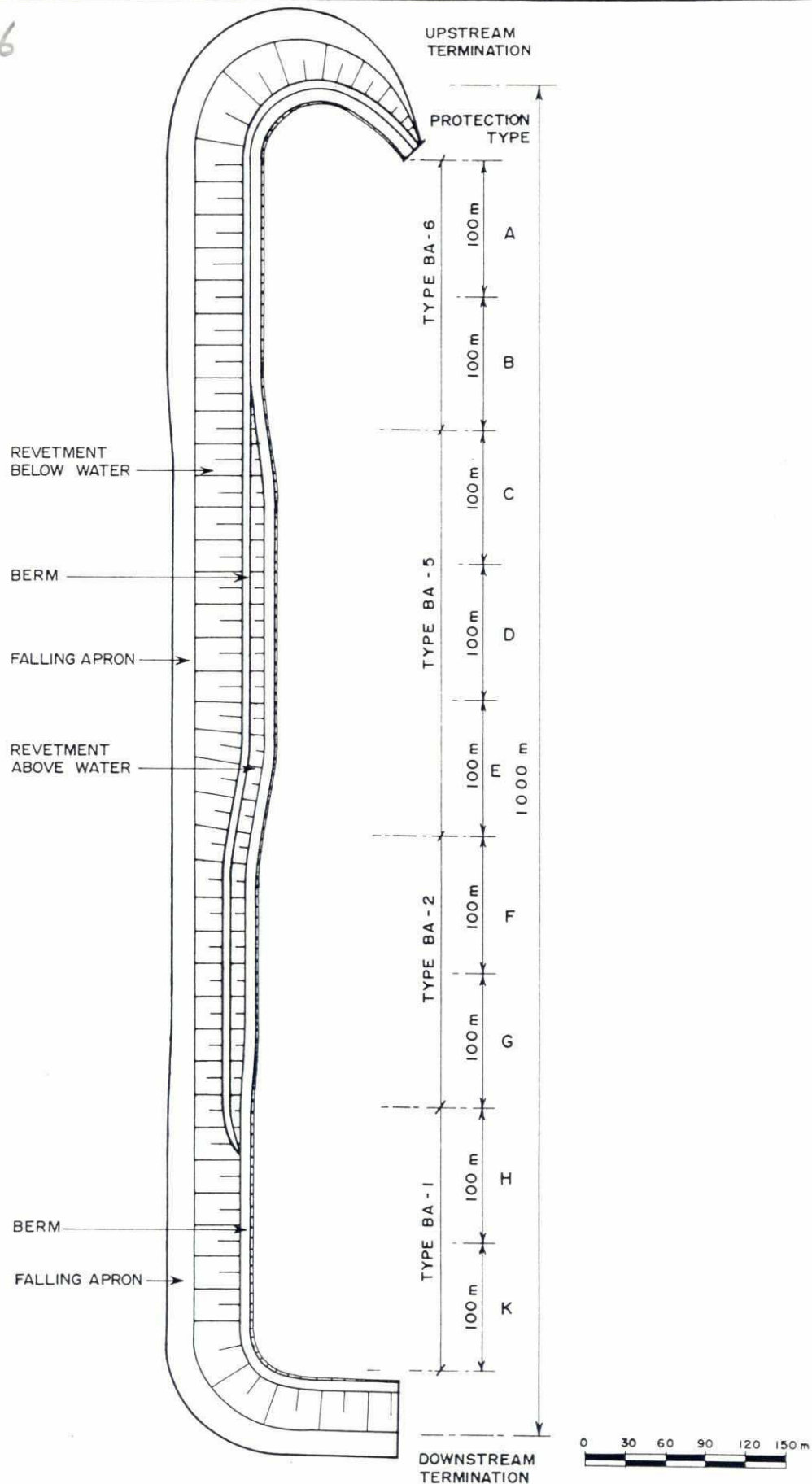
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FIG. A.21-12

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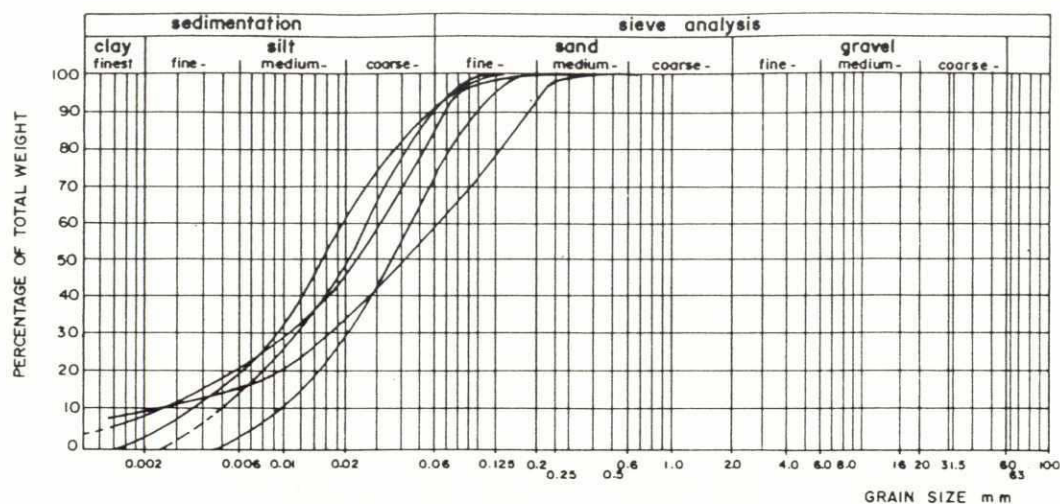


LAYOUT OF EMBANKMENT
AT BAHADURABAD
TEST SITE

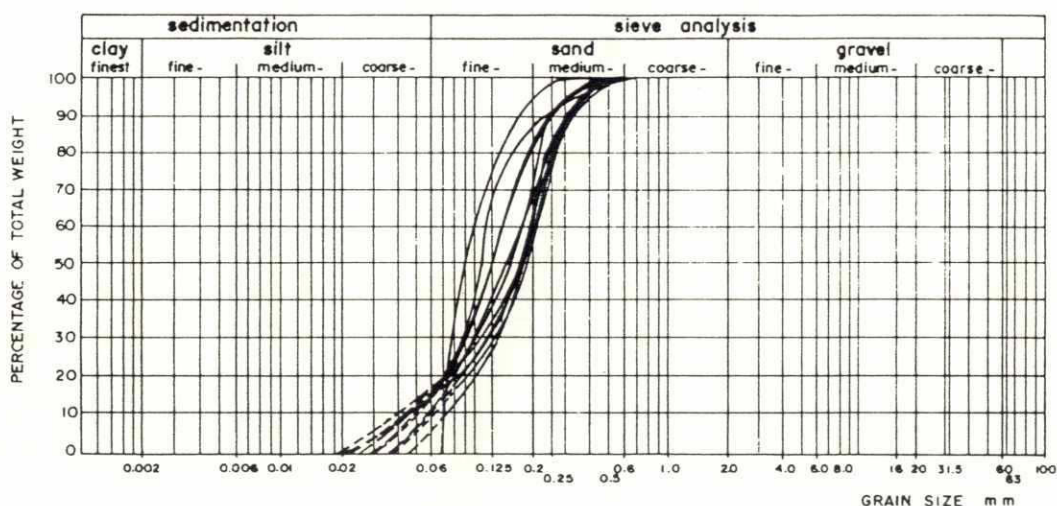
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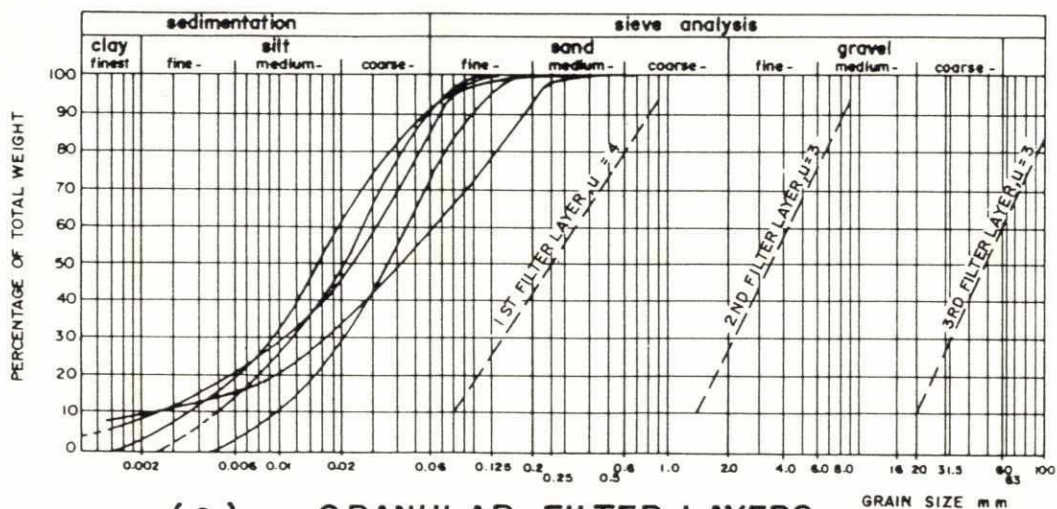
FIG. A. 21 - 13



(a) TYPE OF SOIL DOWN TO ABOUT 5.5 m



(b) TYPE OF SOIL BELOW ABOUT 5.5 m



(c) GRANULAR FILTER LAYERS

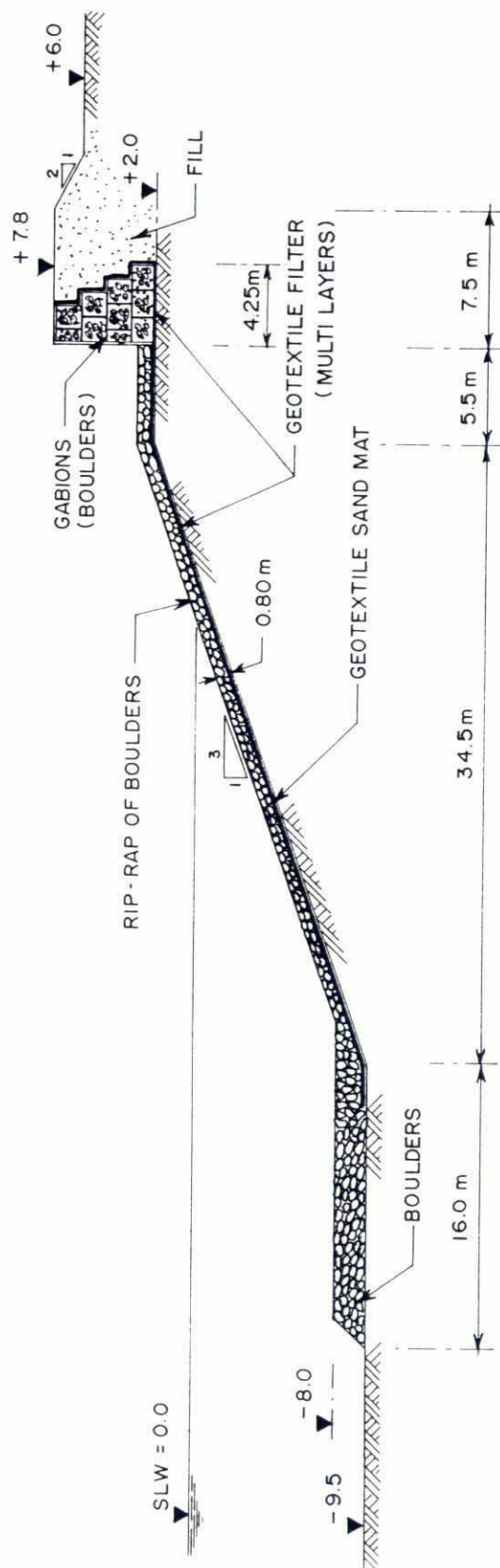
TYPES OF SOIL AND GRANULAR
FILTER LAYERS OF TEST SITE
AT BAHADURABAD

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FIG. A.21- 14

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BOULDERS FOR COVER LAYER
AND FALLING APRON :

$D_{15} > 0.36 \text{ m } (> 115 \text{ Kg})$
 $D_{50} > 0.48 \text{ m } (> 285 \text{ Kg})$
 $D_{100} > 0.61 \text{ m } (> 575 \text{ Kg})$

BOULDERS FOR GABIONS :

$D > 0.17 \text{ m } (> 13 \text{ Kg})$

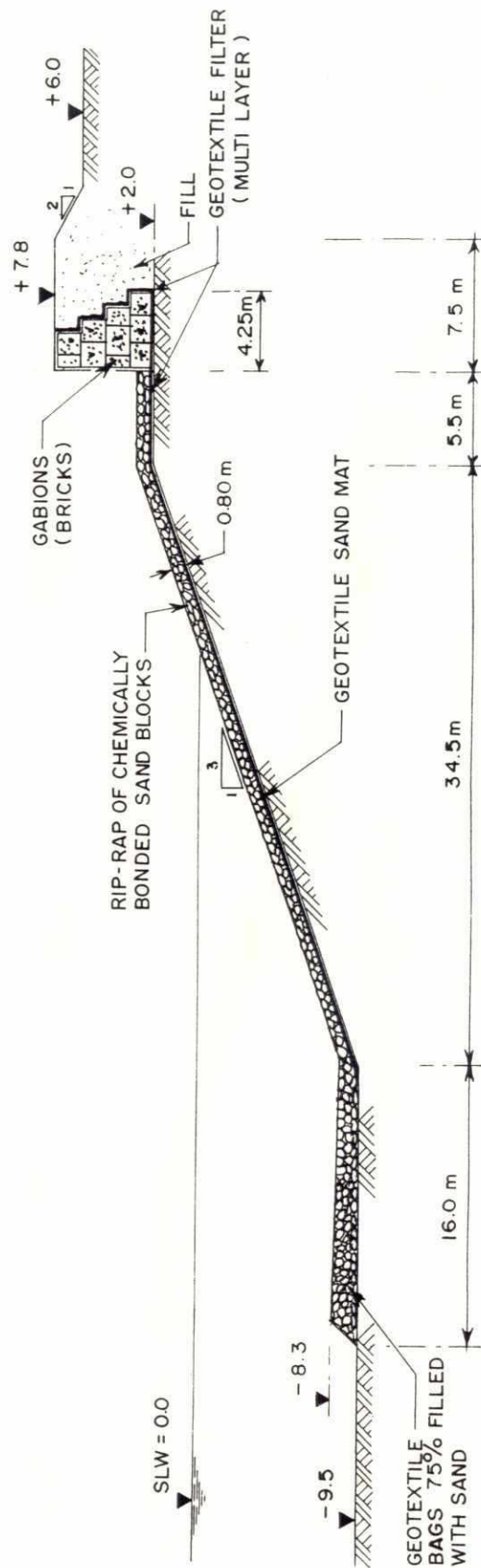
TEST STRUCTURE A
AT BAHADURABAD TEST SITE

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FIG. A. 21- 15



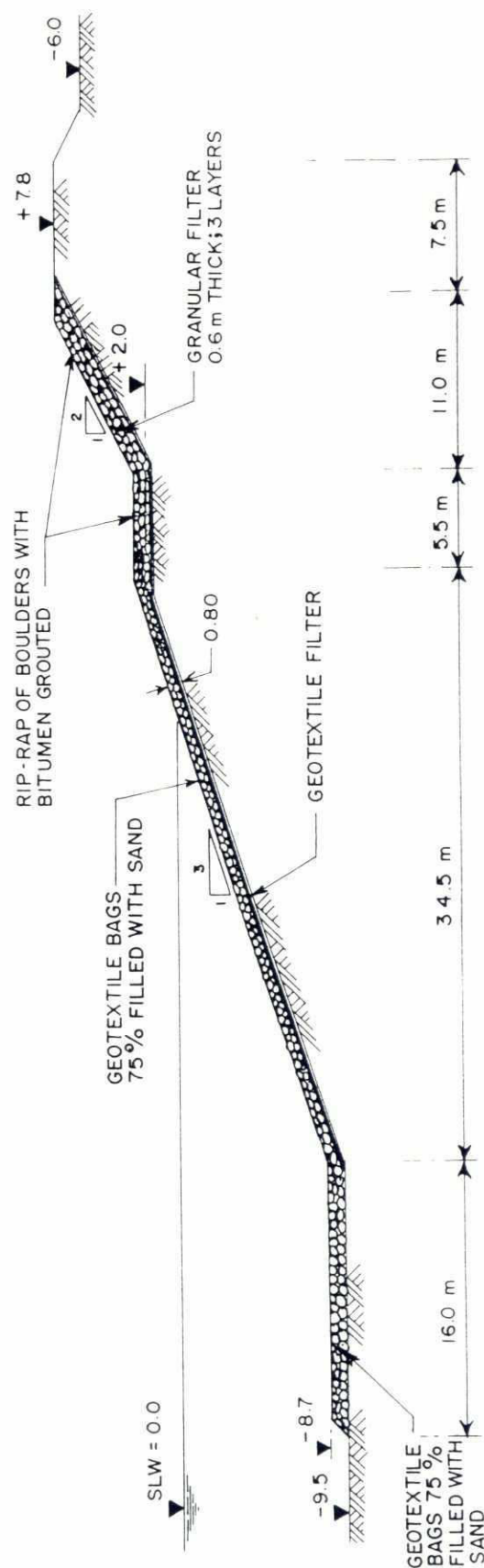
TEST STRUCTURE B AT BAHADURABAD TEST SITE

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FIG.A.21- 16

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BOULDERS FOR COVER LAYER:

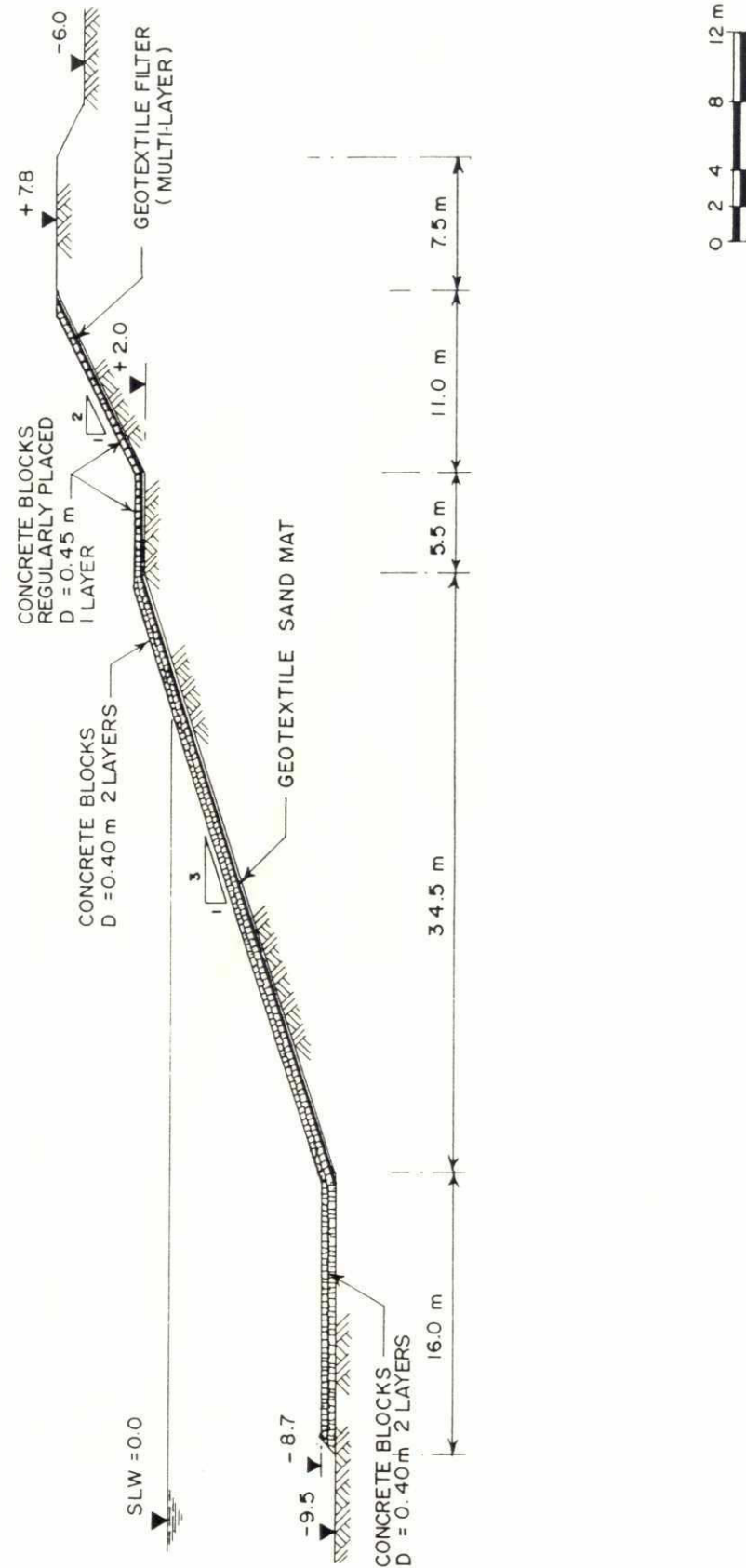
D_{15}	$> 0.27 \text{ m } (> 50 \text{ Kg})$
D_{50}	$> 0.36 \text{ m } (> 120 \text{ Kg})$
D_{100}	$> 0.46 \text{ m } (> 240 \text{ Kg})$

TEST STRUCTURE C
AT BAHADURABAD TEST SITE

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FIG.A.21-17



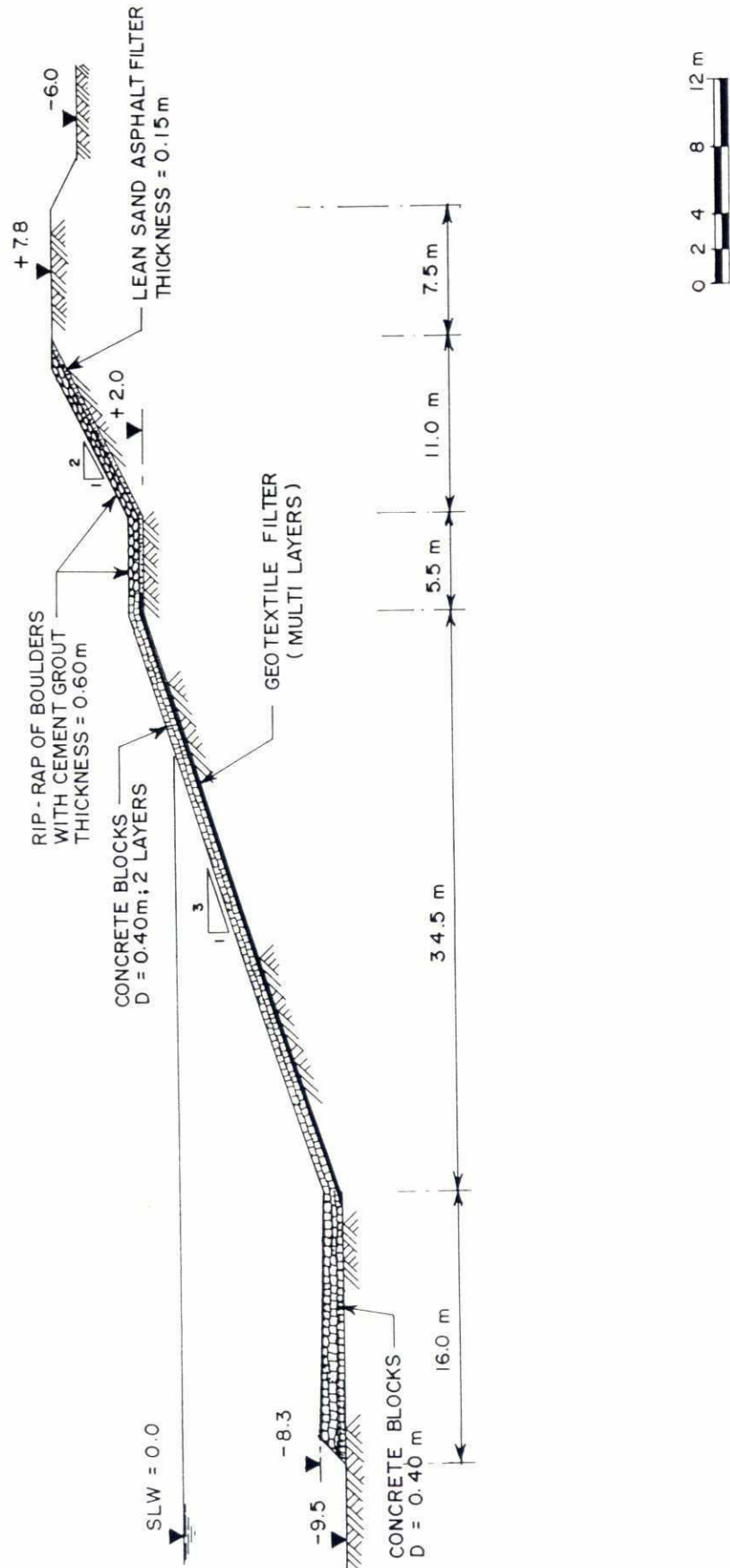
TEST STRUCTURE D
AT BAHADURABAD TEST SITE

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FIG. A.21-18

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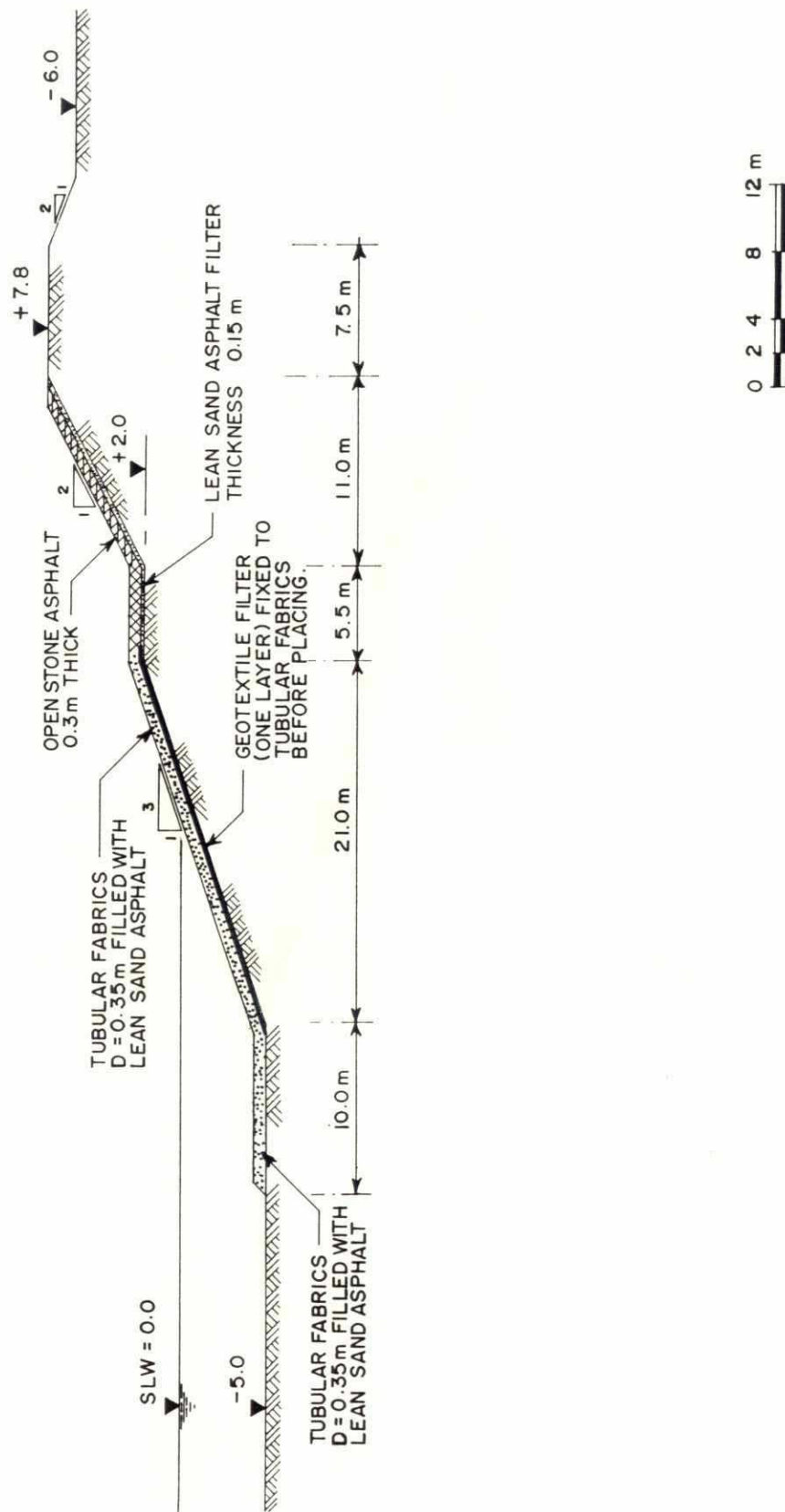
TEST STRUCTURE E
AT BAHADURABAD TEST SITE

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FIG. A. 21-19

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TEST STRUCTURE F AT BAHADURABAD TEST SITE

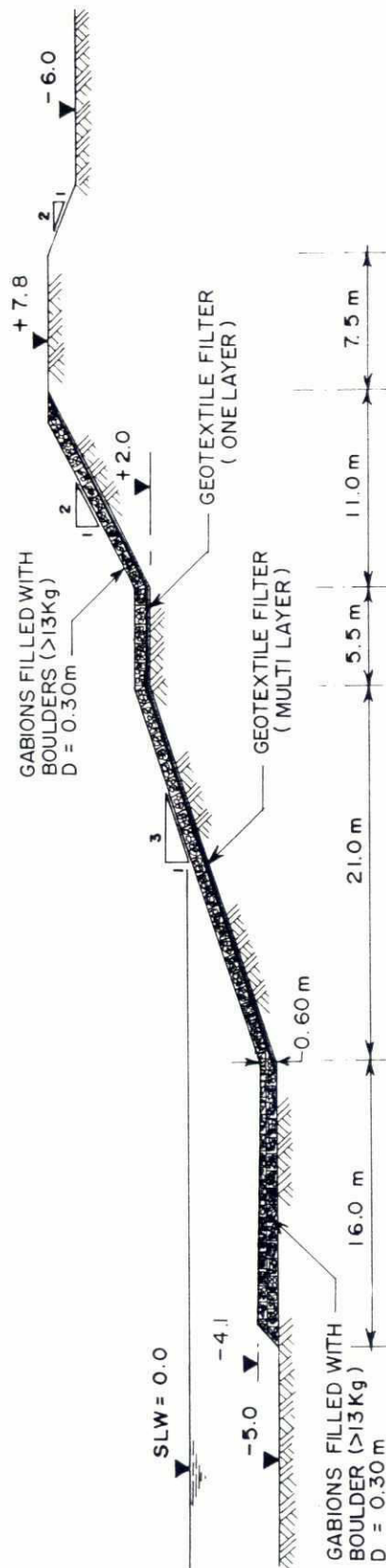
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FIG.A.21-20

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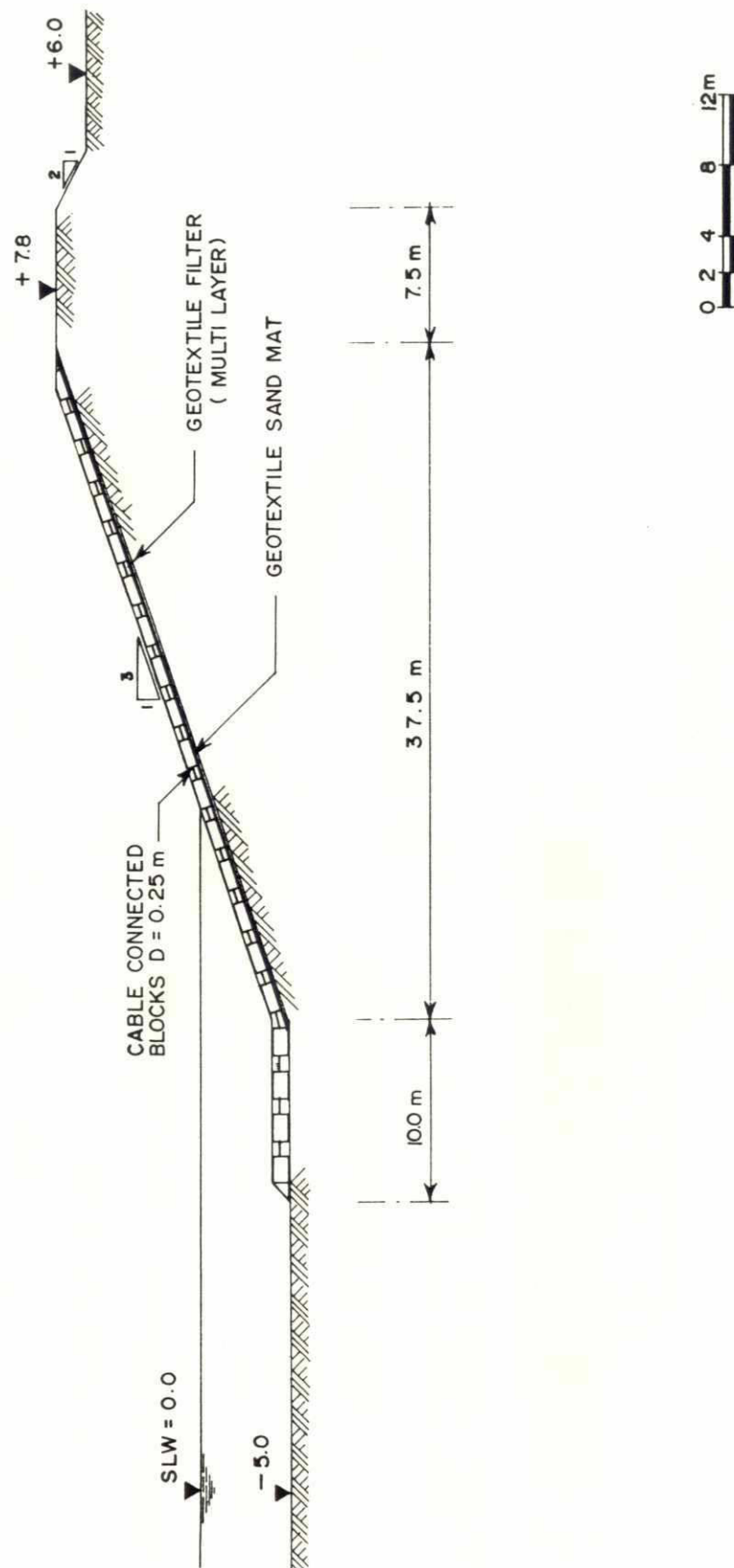
TEST STRUCTURE G AT BAHADURABAD TEST SITE

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FIG. A.21-21



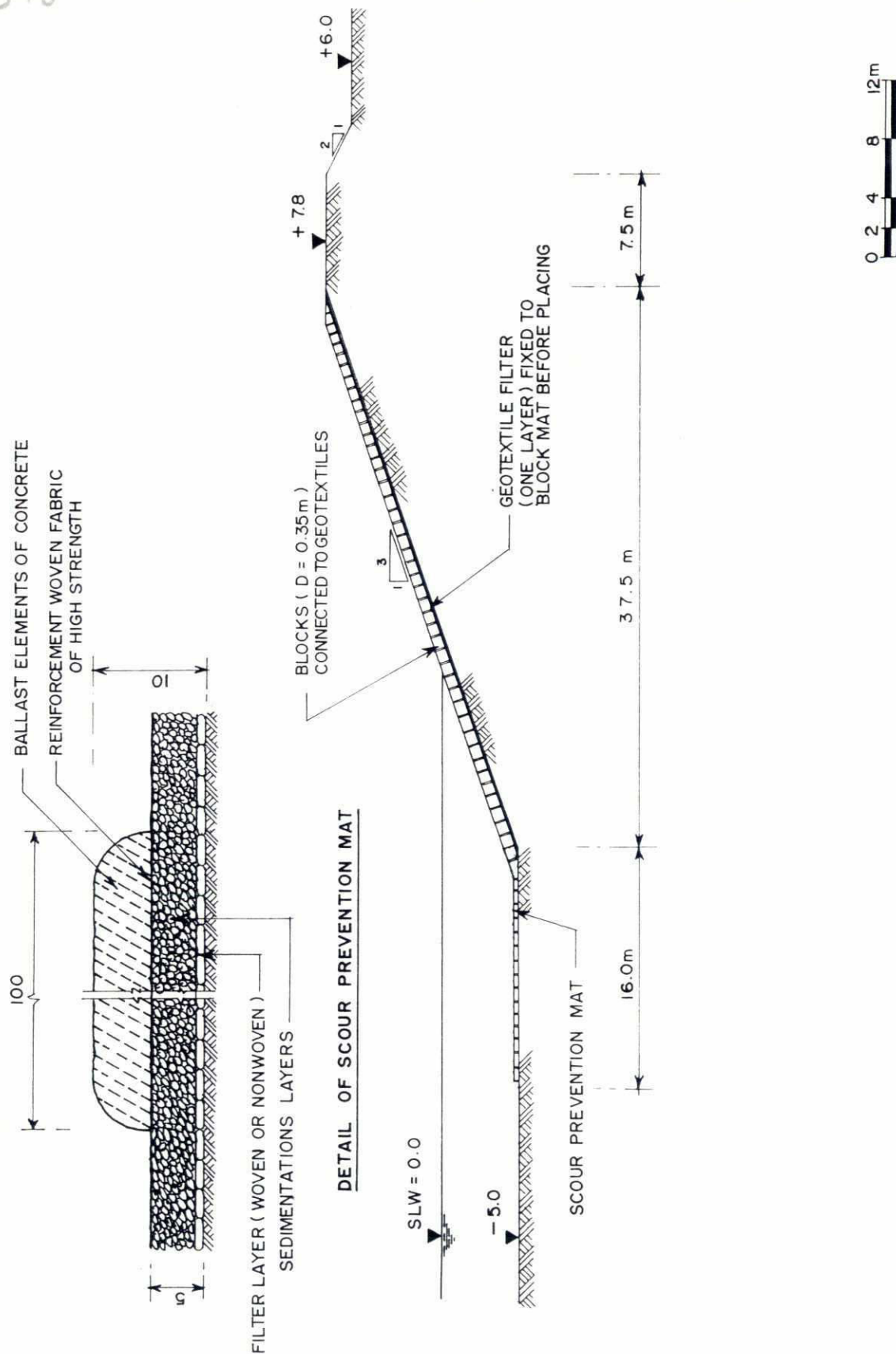
TEST STRUCTURE H
AT BAHADURABAD TEST SITE

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FIG. A.21-22

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TEST STRUCTURE K AT BAHADURABAD TEST SITE

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FIG.A.21-23

**BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22**

**DRAFT FINAL REPORT
PLANNING STUDY**

ANNEX 22

FINANCIAL ANALYSIS

JANUARY 1993

FINANCIAL ANALYSIS

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2.1.1 About the Kamarjani Test Site	2
2.1.2 About the Bahadurabad Test Site	2
2.2 Main Building Components and Unit Costs	2
2.3 Preliminary Cost and Cash Flow of Test Sites	7
3 ECONOMIES OF SCALE	
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3.2 Effects on Kamarjani Type of Works	11
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1 INTRODUCTION

Heavy and erratic erosion of the banks of the Jamuna river is happening from time immemorial and the people gradually got acquainted to it by adapting their lives to the threats by nature. However, during the last decades, mainly originated by the increase of the population and the economic development of the area adjacent to the river the erosion is causing increased damage in lives, property and infrastructures. Therefore, bank protection works have been constructed since several decades to stop the erosion and to protect valuable infrastructure. The methods applied are adapted from other rivers in the area and not fully suited for the particularly difficult conditions of the Jamuna river. As a consequence they are either failing or devouring huge amounts of funds for regular repair and reconstruction.

The present Project is therefore to find technically and economically better solutions. As per the Terms of Reference the Project "aims at evolving appropriate bank protection techniques considering river characteristics, availability of material, construction method and cost involved".

The achieve that goal the approach has to include both a

- technical component for the design and construction method and a
- value analysis to reach an optimum relation of capital and maintenance cost.

At the present stage of the Project the locations for the test structures at Kamarjani and Bahadurabad are tentatively determined according to the best available forecast of the bank erosion which is expected to happen the next one or two monsoon periods. The final design and elaboration of specifications is scheduled for early 1993 and early 1994 for Kamarjani and Bahadurabad, respectively. Hence the estimates of the construction costs are based on a preliminary design standard and are subject to modifications due to possible changes of the locations of the test sites. The estimates of the budget for maintenance, repair and adaptation are hence even more difficult to establish.

For all these reasons the present study can be only a pre-financial analysis in order to :

- identify the components of the construction and adaptation costs;
- analyse the possibilities of scale economy related to big building site, and
- initiate a preliminary value analysis.

2 COST ANALYSIS

2.1 DESCRIPTION OF THE TEST STRUCTURES

The engineering characteristics of the test structures are indicated in Volume I A, Chapter 8 and 9.

2.1.1 About the Kamarjani Test Site

The test structures at Kamarjani will be implemented in the dry season 1993/94. It is proposed to construct 5 permeable groynes with a length of 120 m each and with an orientation of about 15° to the flow direction.

The construction costs may change according to the engineering design which will be adopted according to the test results particularly in matter of:

- the impermeable stretch either as an
 - . earth dam
 - . sheet pile cofferdam
- the permeable stretch either
 - . steel piles or
 - . concrete piles
- the provision of scour material.

The following design criteria for the test structures at Kamarjani were fixed:

standard low water level SWL	:	+ 15.0 PWD
design flood level	:	SWL + 7.5 m
scour depth	:	6 m below river bed
design flow velocity	:	3.0 m/s
significant wave height	:	1.0 m

2.1.2 About the Bahadurabad Test Site

The Bahadurabad test site is located on the left bank south of the present ferry ghat and close to the village of Belgacha.

Various alternatives will be taken into account as far as the three main components of a revetment are concerned, notably:

- the cover layer above and below low water level/ground water level;
- the filter layer above and below low water level/ground water level, and
- the toe protection.

A very important component of the construction and adaptation costs will be related to the level and the dimensions of the falling apron.

The following design criteria for the test structures at Bahadurabad were fixed:

standard low water level SWL	:	+ 13 m PWD
design flood level	:	SWL + 7.5 m
scour depth	:	5 to 11 m below river bed
design flow velocity	:	3.5 m/s
significant wave height	:	1.0 m

2.2 MAIN BUILDING COMPONENTS AND UNIT COSTS

For the present prefinancial analysis the following sources were used:

- schedule of rates of the Water Development Board and the Public Works Department;
- quotations of local firms;
- quotations of foreign firms and importers for imported materials;
- other FAP consultants' estimates, and
- interviews with constructors and BWDB fields engineers.

The proposed proceeding of unit rate analysis builds up on:

- the identification of the items that will be taken into account to estimate the requirements of local and imported materials, labour, equipment or machinery;
- the identification of the sequence of the work in order to adjust the costs based on 1991/92 prices with inflation rates of annually 10% for local and 4% for foreign cost;
- the unit costs expressed at current market price at the construction site if available or increased by the transport cost from the most likely procurement area, and
- the mobilization and organization of construction sites.

Tables 2.2-1 and 2.2-2 show unit costs and percentages of foreign cost of major items for the test sites of Kamarjani and Bahadurabad, respectively. Table 2.2-3 gives the tentative time schedule proposed for the final design, construction, and adaptation of the bank protection tests of Kamarjani and Bahadurabad.

1 Construction Method (Sequence of works)

The Kamarjani test structure will be implemented in the dry season 1993/94. This structure consists of 5 permeable groynes with a length of 120 m each and with an orientation of about 15 nos. to the flow direction. The landward end of the groynes would be impermeable and either be built as a conventional earth dam or as a sheet pile cofferdam.

Description	Unit	Average Unit Cost Tk	Foreign Cost (%)
2. Materials			
2.1 Cement	ton	4,400	90
2.2 Bricks	No	2.5	-
2.3 Sand	m ³	680	-
2.4 Gravel	m ³	930	-
2.5 Steel pipes	ton	55,000	60
2.6 Sled Steel piles	ton	33,000	90
2.7 Fuel	l	15	90
2.8 Lubricant	l	60	90
2.9 Geotextiles	m ²	120	90
3. Labour requirements			
3.1 Unskilled labour	day	60	0
3.2 Mason, Brick player	day	100	0
3.3 Mechanic	day	120	0
3.4 Peadman	day	140	0
3.5 Foreman	day	200	0
3.6 Surveyor	day	250	0
4. Machinery and equipment requirements			
4.1 Local hire }	day	85,000	0
4.2 Local purchase }			
4.3 Foreign hire }	day	300,000	100
4.4 Foreign purchase }			
5. Transport	ton x km	750	

Table 2.2-1: Cost analysis, description and components for Kamarjani

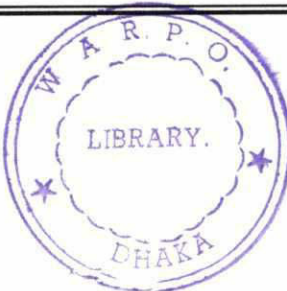
1 Construction Method (Sequence of works) The implantation of the test structure is planned for the dry season 1994/95: - the cover layer - the filter layer - the toe protection			
Description	Unit	Average Unit Cost Tk	Foreign Cost (%)
2. Materials 			
2.1 Cement	ton	4,400	90
2.2 Bricks	No	2.5	-
2.3 Sand	m ³	680	-
2.4 Gravel	m ³	930	-
2.5 Steel pipes	ton	33,000	60
2.6 Fuel	l	15	90
2.7 Lubricant	l	60	90
2.8 Geotextiles	m ³	1,500	90
3. Labour requirements			
3.1 Unskilled labour	day	60	0
3.1 Semi skilled	day	100	0
3.2 skilled	day	120	0
3.3 Mason/Mechanic	day	140	0
3.4 Foreman	day	200	0
3.5 Surveyor	day	250	0
3.6 Diver	day	600	0
4. Machinery and equipment requirements			
4.1 Local hire }	day	85,000	10
4.2 Local purchase }			
4.3 Foreign hire }	day	300,000	100
4.4 Foreign purchase }			
5. Transport	ton x km	750	

Table 2.2-2: Cost analysis, description and components of Bahadurabad

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A22 - 6

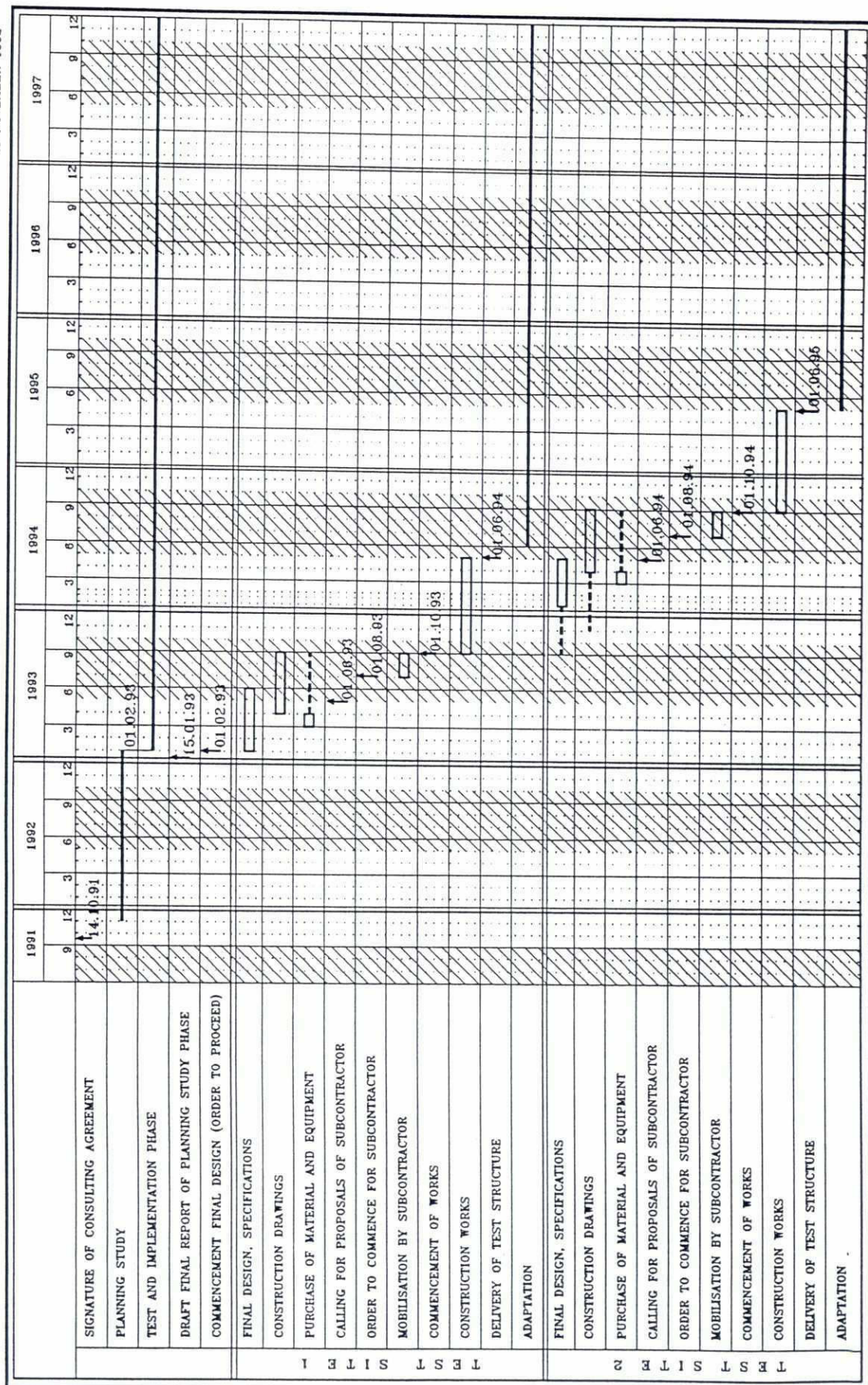
TABLE 2.2-3

FAP 21 TEST STRUCTURES

PRELIMINARY TIME SCHEDULE FOR FINAL DESIGN, CONSTRUCTION AND ADAPTATION

VERSION NO 1

20 NOVEMBER 1992



MONSOON



2.3 PRELIMINARY COST AND CASH FLOW OF TEST SITES

A general breakdown of major cost components for construction and adaptation as estimated for the test sites at Kamarjani and Bahadurabad is given in Table 2.3-1 and Table 2.3-2, respectively. Due to the particular conditions of test sites many alternative items will be built in small quantities. The unit rates which these cost estimations are based on, took that fact into account.

Sl.No.	Test site 1/Kamarjani	Tk
1.	Imported by Consultant	138,575,000
2.	Local purchase by Consultant	79,029,000
3.	Subcontract	154,781,200
4.	Contingencies	48,514,800
	Total test site 1	420,900,000
	Test site 2/Bahadurabad	
1.	Imported by Consultant	14,950,000
2.	Subcontract including local purchase	324,050,000
3.	Contingencies	63,500,000
	Total test site 2	402,500,000

Cost based on 1991/1992 prices inflated by annually 10% for local and 4% for foreign cost.

Table 2.3-1: Preliminary construction costs of test structures

Item	1,000 Tk	%
Kamarjani		
- imported equipment	4,600	0.8
- Adaptation	92,920	75.4
- Contingencies	28,980	23.8
Total	121,900	100
Bahadurabad		
- Imported equipment	5,750	0.6
- Adaptation	66,010	79.4
- Contingencies	17,940	20
Total	89,700	100

Table 2.3-2: Monitoring/adaptation costs

The adaptation costs include the following main components

-	Local materials and labour	:	30%
-	Labour	:	7%
-	Mobilization	:	16%
-	Purchase and hired equipment	:	23%
-	Transport	:	1%
-	Contractors' overheads and profit	:	23%

A preliminary assessment of the cost estimates and the cash flow of the test works is given in Table 2.3-3 and 2.3-4 for Kamarjani and Bahadurabad, respectively.

		Tk	DM	DATE
1	IMPORTED BY CONSULTANT			
	Equipment	69,000,000	3,000,000	01-04-93
	Steel plates for piles	62,100,000	2,700,000	15-04-93
	Geotextiles bags and filter	2,875,000	125,000	15-04-93
	Monitoring equipment	4,600,000	200,000	01-01-94
	TOTAL ITEM 1	138,575,000	6,025,000	
2	LOCAL PURCHASE BY CONSULTANT			
	Steel piles (rolling + transport)	64,000,000	2,782,600	01-07-93
	Concrete piles (production + transport)	11,904,000	517,600	01-07-93
	Equipment transport	3,125,000	135,900	01-07-93
	TOTAL ITEM 2	79,029,000	3,436,100	
3	SUBCONTRACT			
	Advance payment	26,248,000	1,141,200	01-08-93
	1st installment	22,540,000	980,000	01-10-93
	2nd installment	22,540,000	980,000	01-12-93
	3rd installment	22,540,000	980,000	01-02-94
	4th installment	22,540,000	980,000	01-04-94
	5th installment	22,540,000	980,000	01-06-94
	Final installment	15,833,200	688,400	01-08-94
	TOTAL ITEM 3	154,781,200	6,729,600	
4	CONTINGENCIES	48,514,800	2,109,300	
	TOTAL ITEM 4	48,514,800	2,109,300	
	TOTAL TEST SITE 1	420,900,000	18,300,000	

Observations :

- 1) Exchange rate : DM 100 equal to Tk 2,300
- 2) Costs based on 1991/1992 prices inflated by annually 10% for local and 4% for foreign cost

Table 2.3-3: Preliminary cost and cash flow of test site 1 : Kamarjani

		Tk	DM	DATE
1	IMPORTED BY CONSULTANT			
	Geotextiles (geosynthetics)	9,200,000	400,000	01-04-94
	Monitoring equipment	5,750,000	250,000	01-01-95
	TOTAL ITEM 1	14,950,000	650,000	
2	SUBCONTRACT INCL. LOCAL PURCHASE			
	Advance payment	39,450,000	1,715,200	01-08-94
	1st installment	31,050,000	1,350,000	01-10-94
	2nd installment	31,050,000	1,350,000	01-11-94
	3rd installment	31,050,000	1,350,000	01-12-94
	4th installment	31,050,000	1,350,000	01-01-95
	5th installment	31,050,000	1,350,000	01-02-95
	6th installment	31,050,000	1,350,000	01-03-95
	7th installment	31,050,000	1,350,000	01-04-95
	8th installment	31,050,000	1,350,000	01-05-95
	Final installment	36,200,000	1,573,900	01-08-95
	TOTAL ITEM 3	324,050,000	14,089,100	
3	CONTINGENCIES	63,500,000	2,760,900	
	TOTAL ITEM 4	63,500,000	2,760,900	
	TOTAL TEST SITE 2	402,500,000	17,500,000	

Observations :

- 1) Exchange rate : DM 100 equal to Tk 2,300
- 2) Costs based on 1991/1992 prices inflated by annually 10% for local and 4% for foreign cost

Table 2.3-4: Preliminary cost and cash flow of test site 2 : Bahadurabad

3 ECONOMIES OF SCALE

3.1 RELATIVE WEIGHT OF INDIVIDUAL COST COMPONENTS

Estimating the cost of each component of the bank protection works in terms of percentage of the total construction costs, Table 3.1-1 and 3.1-2 show the relative weight of each component in relation to the total construction cost.

The tables show that the most important costs components are related to:

- local and foreign equipments and special materials and
- mobilization.

In these tables the economies of scale attainable for main cost components are related to the latters' contribution to a total cost reduction in the case of large scale works.

3.2 EFFECTS ON KAMARJANI TYPE OF WORKS

Table 3.1-1 shows that imported special materials and equipments represent an important part of the costs (33%) followed by local purchases (about 19%) and mobilization (12%). On the contrary labour represents a relative smaller part of the costs.

3.3 EFFECTS ON BAHADURABAD TYPE OF WORKS

For this type of works (see Table 3.1-2), materials and purchased and hired equipments represent a cost part a little less important than in the case of the corresponding items of the Kamarjani site (49% maximum). Mobilization represents around 12% of the total cost.

Item	Cost components	Contri- bution to total cost (in %)	For large construction works	
			Maximum margin of variation due to economy of scale (in %)	Relative margin of variation (in %)
1.	Imported			
	- cement	4.3	- 15	- 0.6
	- equipment steel plates for piles	12.2	- 28	- 3
	- geotextiles	14.8	- 15	- 2.2
	- monitoring	0.7	- 15	- 0.1
	- equipment	1.0	-	-
	Sub-total	33.0		- 5.9
2.	Local purchase			
	- steel piles	15.2	- 20	- 3
	- concrete piles	2.8	- 20	- 0.6
	- equipment transport	0.7	- 20	- 0.1
	Sub-total	18.8		- 3.7
3.	Sub-contract			
	- other local materials	5.5	- 30	- 1.7
	- unskilled labour	1.6	- 10	- 0.1
	- labour	0.2	- 15	-
	- mobilization	11.8	- 30	- 3.5
	- contractor's overhead incl. profit and risk	17.6	-	- 2.6
	Sub-total	36.7		- 7.9
4.	Contingencies	11.5		- 3
	Total Test Site 1	100.0		- 20.5

Table 3.1-1: Cost reduction by economy of scale (Kamarjani)

Item	Cost components	Contri- bution to total cost (in %)	For large construction works	
			Maximum margin of variation due to economy of scale (in %)	Relative margin of variation (in %)
1.	Imported			
	- cement	2.4	- 15	- 0.4
	- geotextiles and other materials	0.9	- 15	- 0.1
	- monitoring equipment	0.4	-	-
	Sub-total	3.7		- 0.5
2.	Local purchase			
	- local materials	25.2	- 20	- 5
	- labour unskilled	4.4	- 10	- 0.4
	skilled	1.0	- 15	- 0.2
	- mobilization, equipment and machinery*	32.7	- 30	9.8
	- contractor's overhead incl. profit and risk	17.2	-	- 2.6
	Sub-total	80.5		- 18
3.	Contingencies	5.2		- 3
	Total Test Site 2	100.0		- 21.5

* hired and purchased

Table 3.1-2: Cost reduction by economy of scale (Bahadurabad)

4 COSTS SENSITIVITY ANALYSIS AND CONCLUSION

A tentative sensitivity analysis of the costs has been carried out on the basis of the cost components as indicated in Table 3.1-1 and 3.1-2. The potential different factors on possible cost reduction is qualitative shown in Table 4-1.

Significant factors	Cost components							
	Imported materials		Imported equipments		Local purchase		Labour	Mobilization
	Qty.	Price	Qty.	Price	Qty.	Price		
Economy of scale	+++	+	+++	+	+++	++	+	+++
Coefficient of equipment efficiency			++					
Construction management			++					+++
Engineering design	++				++			
Construction technique						++		

Table 4-1: Potential of different factors on a possible cost reduction

Relating the potential of different factors on possible cost reduction to their contribution to the total cost about 20 % overall cost reduction - as compared to the test structures - seems to be attainable.

The most significant possible variations related to a groyne structure comparable with the Kamarjani test site concern essentially:

- imported special material and equipment (particularly steel plates for piles);
- local purchase like steel pile, and
- mobilization costs.

In the case of revetments similar to those of the Bahadurabad test site the most important variations concern:

- local materials and
- the mobilizations costs.

So, in order to improve both the cost effectiveness and the relation of capital to maintenance and adaptation cost it will be necessary:

(1) for material purchases

- to get negotiated prices according to the quantity of materials ordered and
- to test effective engineering designs.

(2) for hired and purchased equipment

- to get negotiated prices according to the equipment required and
- to develop an effective construction management in order to reduce the duration of the site and to achieve a high equipment efficiency.

(3) for mobilization

- to reduce the costs taking into account possible economies of scale and
- to carry out an effective construction management of this specific phase.

All these suggestions are also true for maintenance and adaptations works.

To sum it up, three important issues have to be taken special care of:

- (i) the test structures shall find ways to make use of the benefits of economics of scale when recommending standard designs for the future;
- (ii) the large potential of future cost reductions suggest a close cooperation between engineers and economists during the Test and Implementation Phase, and
- (iii) after having the experiences of the construction as well as the adaptation of the test structures a detailed sensitivity analysis is intended to be carried out.

