GOVERNMENT OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANIZATION

Bank Protection Pilot Project FAP 21

Guidelines and Design Manual for Standardized Bank Protection Structures

December 2001

2

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We hope this handbook contributes a small step towards systematic and improved strategies on planning and implementation procedures as well as towards optimized solutions for a safe and economic design of protection structures against bank erosion at Bangladesh rivers.

December 2001

Guidelines and Design Manual for Standardized Bank Protection Structures

Table of Contents

LIST OF ACRONYMS

ACM	-	Articulating Concrete Mattress
ADB	-	Asian Development Bank
AFP	-	Active Flood Plain
AFPM	-	Active Flood Plain Management
ASTM	-	American Society for Testing Materials
BIWTA	-	Bangladesh Inland Water Transport Authority
BIWTC	-	Bangladesh Inland Water Transport Corporation
BS	-	British Standards
BTM	-	Bangladesh Transverse Mercator (Projection)
BUET	-	Bangladesh University of Engineering and Technology
BWDB	-	Bangladesh Water Development Board
CC	-	Cement Concrete
CPT	-	Cone Penetration Test
CV	-	Curriculum Vitae
DGPS	-	Differential Global Positioning System
DHI	-	Danish Hydraulic Institute
DHW	-	Design High Water
DIN	-	Deutsche Industrie Norm (i.e. German Industrial Standard)
DWL	-	Design Water Level
EDM	-	Electronic Distance Meter
EGIS	-	Environment and GIS Support Project for Water Planning Sector
EPC	-	Engineering-Procurement-Construct
ERD	-	Economic Relation Division
FAP	-	Flood Action Plan
FIDIC	-	Fédération Internationale des Ingénieurs-Conseils
FPCO	-	Flood Plan Coordination Organization
FPL	-	Flood Plain Level
F_s	-	Factor of Safety
GIS	-	Geographic Information System
GoB	-	Government of Bangladesh
GPS	-	Global Positioning System
ICB	-	International Competitive Bidding
ISBN	-	International Standard Book Number
LCB	-	Local Competitive Bidding
MCA	-	Multi-Criteria-Analysis
NGO	-	Non-Governmental Organization
PDR	-	Preliminary Design Report
PES	-	Polyester
PIANC	-	Permanent International Association of Navigation Congresses
PP	-	Polypropylene
PPS	-	Project Planning Schedule
PQ	-	Pre-qualification

PVC	-	Polyvinyl Chloride
PWD	-	Public Works Department (datum level)
RRI	-	River Research Institute
SC	-	Structure Category
SHW	-	Standard High Water
SIPPS	-	Strategy for Identification of Priority Protection Sites
SLW	-	Standard Low Water
SMP	-	Standard Master Plan
SPOT	-	System Probatoire d'Observation de la Terre
SPS	-	Standardized Protection Structures
SPT	-	Standard Penetration Test
SWG	-	Standard Wire Gauge
SWL	-	Still Water Level
SWMC	-	Surface Water Modelling Centre
WARPO	-	Water Resources Planning Organization

GLOSSARY

Alluvial	describing the genesis of sediments by flow of rivers
Angle of flow approach	local angle between direction of approaching flow and bankline
Axis-symmetrical	related to idealized bends with uniform curvature
Bed protection	layered systems on filters placed at a horizontal bed as protection against hydraulic forces and scouring
Bend scour	scour in outer bend 
Braiding	formation of a river course with multiple channels, divided by bars with a size in the order of the channel width 
Char	island, sand bank and floodable area adjacent to the banks
Chézy coefficient	coefficient of hydraulic roughness used for free surface flows
Common excavation	stripping, excavation or removal of any type of material on or near bank for construction pits or embankment revetments and bed protections, whether in dry or in wet condition
Confluence scour	scour created by turbulences when two channels or rivers join 
Conveyance	geometrical property of a cross-section which determines the relation between discharge and cross-sectional averaged flow velocity
Cover layer	outer protective layer of an embankment revetment or a bed protection
Cross bar	impermeable structure protruding into the river flow
Cushion layer	intermediate layer of an embankment revetment or a bed protection, i.e. the layer above a filter or sublayer and below the cover layer
Dredging	removal of any soil by bank-sided or floating equipment below water level, irrespective of the method employed
Earthworks	"common excavation" and "filling works"

Expansion angle	angle between main flow direction and the flow line behind an obstacle due to flow separation (mixing layer which develops downstream e.g. from a groyne head)
Fabric containers	synthetic fabric bags, mattresses or tubes filled various materials such as sand, bitumen sand, lean sand asphalt etc.
Falling apron	toe protection of granular material, such as concrete blocks or boulders, placed directly on the existing subsoil or river bed (i.e. without filter)
Fetch	length of uninterrupted contact between water surface and atmosphere, allowing water waves to grow by transfer of wind energy
Filling	filling by suitable material and compaction of any land, for construction of embankments or groynes, filling and compacting of construction pits and excavations, back filling of structures, etc., to designed levels
Filter	one-layer or multi layer system of well graded granular material or a geotextile or a combination of both
Flow slide	bank erosion by means of relatively fast mass failure (liquefaction), resulting in deep bays in the bankline with a narrow neck
Gabions	mattresses and rectangular baskets made from protected steel wire mesh and filled with loose material such as boulders, bricks etc.
Geo-textile	synthetic fabric (woven, non-woven, needle punched) applied as a filter or used in tailored geo-textile systems (mattresses, etc.)
Groyne	impermeable or permeable structure protruding into the river flow
Hardpoint	local in-erosive bankline either natural or artificial (massive, stable structure)
Hydraulic loads	forces due to action of water (hydrostatic or hydrodynamic)
Impinging flow	flow attacking e.g. a river bank
Interaction scour	scour enhancement due to the superposition of different scour mechanism, creating larger total scour depth as compared to the theoretical summation of the individual scour depth
Khoa	brick chips, used as concrete aggregates and filter material
Launching apron	integrated and articulating toe protection, i.e. mattress systems, such as sand-filled geo-textiles, concrete-filled geo-textiles or concrete blocks linked to a strong geo-textile, placed on prepared slopes and a filter layer above and below water or in a horizontal excavation above SLW.
Lifetime	total time for which the structure is designed to remain in function
Local scour	scour downstream from a local structure or obstruction

Meandering	formation of a sinuous river course through bank erosion 
Mixing layer	transition zone at the interface of two parallel flows of different velocity, produced by turbulent mixing
Mouza	official land owner map
Opening size	the dimension which corresponds to the average size of particles of which n % by weight are able to pass through a geotextile fabric
Oversteepening	Erosion induced process at the bank toe creating a very steep and unstable bank
Peak flow	flow event where maximum flow velocities are observed. Peak flows do not necessarily coincide with maximum water depths
Permittivity	the permeability of a geo-textile normal to the fabric per unit thickness of the fabric
Planform	shape on map of banklines or water lines (top view)
Point bar	bar at inner bank of a river bend
Protrusion scour	scour immediately upstream from a local structure or obstruction due to local acceleration of the flow 
Regime equations	empirical formulae based on typical relations between channel dimensions (incl. slope and roughness) and river discharge
Return period	recurrence time, average time interval between subsequent events in which conditions are exceeded. When designing a structure, the return period is usually larger than the projected lifetime, because, for instance, if both would equal 50 years, the structure would have a 64% probability of failure during its lifetime
Revetment	layered systems of cover intermediate and filter layers placed on a sloping surface as protection against hydraulic forces and scouring
Rip-rap	layer of loose stones acting as cover layer in an embankment revetment, a bed protection or a falling apron
Rock	any hard natural or artificial material requiring the use of blasting or mechanical tools for its removal
Scour	local erosion of soil particles by current or wave induced shear forces (also "scour hole")
Scour velocity	rate of deepening of the deepest point of a scour hole
Seepage	movement of water into or out of the river bank
Service time	period within the lifetime in which the structure is exposed to attack by the river
Shear failure	normal soil-mechanical slide
Shear stress	force per unit area exerted by the flow on a parallel surface

Shields parameter	parameter expressing the mobility of particles considering the grain size and the prevalent flow conditions
Shoaling	effect of shallow water on waves
Slide	collapse of large portions of a river bank typically induced by excessive pore pressure and top loads (mass failure)
Soil migration	the transportation of the finer soil particles within a soil mass
Spur	local expression for groyne
Sub-layer	any layer between cover layer and filter
Subsoil	naturally deposited or filled and compacted soil material on which an embankment revetment, a bed protection or a falling apron is constructed
Toe protection	systems to protect the toe of an embankment against instability due to erosion/scouring
Top soil	top layer of soil containing a higher proportion of organic material

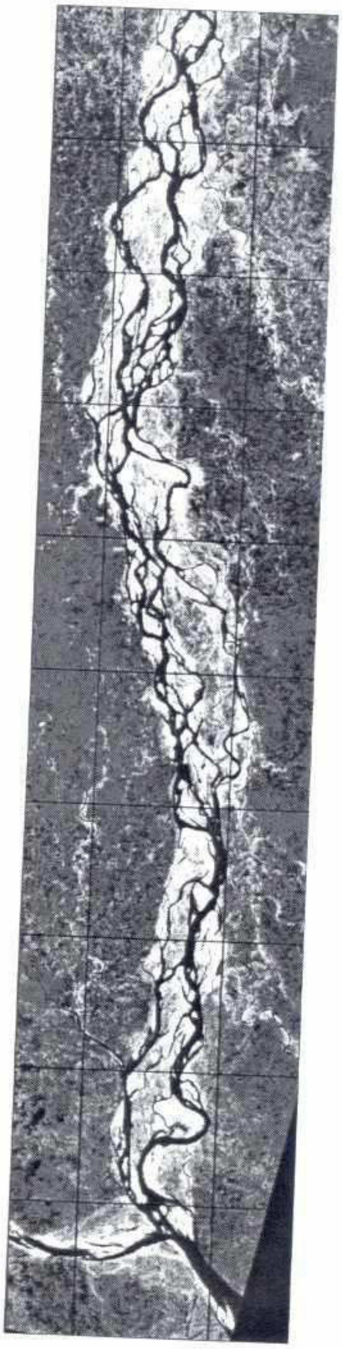
LIST OF SYMBOLS

A	-	cross-sectional area of river	(m^2)
B	-	channel or river width at water surface	(m)
C	-	Chézy coefficient for hydraulic roughness	($m^{1/2}/s$)
c	-	cohesion	(kN/m^2)
c'	-	effective cohesion	(kN/m^2)
D	-	diameter or thickness of protection unit	(m)
D_n	-	grain size diameter corresponding to n % by mass of finer particles	(mm)
e	-	distance between pile axes	(m)
F	-	force	(kN)
Fr	-	Froude number	(-)
g	-	acceleration due to gravity	(m/s^2)
H_{des}	-	design wave height	(m)
H_s	-	significant wave height	(m)
h	-	(local) water depth	(m)
I	-	water level gradient	(-)
i	-	hydraulic gradient in soil	(-)
K_h	-	depth factor	(-)
K_s	-	slope factor	(-)
K_t	-	turbulence factor	(-)
k	-	wave number	(1/m)
k_g	-	permeability of geotextile	(m/s)
k_s	-	Nikuradse sand equivalent coefficient of roughness	(m)
L	-	length of pile	(m)
L_0	-	wave length in deep water	(m)
L_f	-	fetch length	(m)
n	-	cotangent of transverse bed slope	(-)
O_n	-	opening size of a geotextile	(μm)
P	-	permeability of groynes	(-)
Q	-	water discharge	(m^3/s)
q	-	specific discharge	(m^3/sm)
R	-	hydraulic radius	(m)
r	-	co-ordinate along bend radius	(m)
S_G	-	spacing between groynes	(m)
T	-	wave period	(s)
t	-	time	(s)
t	-	wall thickness	(mm)
t	-	time duration	(s)
U	-	circumference (of piles)	(cm)
u	-	depth-averaged velocity	(m/s)
u_1	-	depth-averaged flow velocity at upstream boundary of control	(m/s)
$\bar{u}$	-	cross-sectional and depth-averaged flow velocity	(m/s)
u_b	-	bottom velocity	(m/s)
V_{fa}	-	volume of falling apron per linear metre protected bankline	(m^3/m)

W_{fa}	-	width of falling apron	(m)
y_s	-	maximum local scour depth	(m)
$y(t)$	-	time-dependent local scour depth	(m)
γ_s	-	specific weight of solids	(kN/m^3)
γ_w	-	specific weight of water	(kN/m^3)
γ'	-	specific weight of solids in submerged condition	(kN/m^3)
Δ	-	relative submerged sediment density	(-)
ΔH	-	head loss	(m)
Δp	-	pressure gradient	(kN/m^2)
ε_s	-	angle of repose	(degree)
ρ_s	-	density of protection material	(kg/m^3)
ρ_w	-	density of water	(kg/m^3)
σ	-	total normal stress	(kN/m^2)
σ'	-	effective normal stress	(kN/m^2)
τ	-	shear strength	(kN/m^2)
τ_{mf}	-	skin friction	(kN/m^2)
φ	-	angle of internal friction	(degree)
φ'	-	effective angle of internal friction	(degree)
Ψ_{cr}	-	critical Shields parameter for initiation of motion	(-)

Other symbols are explained in the text at their utilisation.

26



PART 1

GUIDELINES

PART 1 - GUIDELINES

Contents

A

33

8\.

Figures

2 7

Tables

1 INTRODUCTION

1.1 MOTIVATION AND OBJECTIVES

The need of river training and bank protection in Bangladesh arises from the fact that most rivers of the country are unstable, i.e. they are not in a state of equilibrium with the governing physical processes. Unstable rivers undergo permanent and rapid changes of the water and sediment regime and, hence, adjustment in depth, slope, width and planform. Both, river training and bank protection measures, which are strongly interrelated, have the objective to ensure a safe and efficient transport of water and sediments (suspended material and bed load) through a certain defined stretch of the river. A safe and expeditious passage of flood discharge must be possible during high water stages with a minimum of negative effects on surrounding areas.

In order to prevent or minimize the loss of valuable land, several stretches of the river banks might need suitable protection against erosion. In this context it has to be emphasised, that in case local protection measures are planned, also the consequences must be taken into account, because a certain response of the river, i.e., morphological changes in the vicinity or even further away from a countermeasure, are to be expected. Protection of one place will possibly influence erosion and bankline shifting at other locations. From that point of view, only absolutely necessary measures should be considered.

The provision of suitable tools for the planning, design and implementation as well as for appropriate monitoring and maintenance schemes of standardized bank protection structures in Bangladesh are the main objectives of this handbook. The provided information is primarily based on the findings of the Bank Protection Pilot Project (FAP 21) and the River Training/Active Flood Plain Management Pilot Project (FAP 22), but is also including previous experience gained on rivers in Bangladesh and on similar rivers around the globe within other projects.

The book is structured in a way that the concerned planning and design engineers as well as monitoring and maintenance teams are guided to arrive at safe and economic solutions. In Part I – Guidelines, Chapter 1, a brief description of the existing types of rivers as well as of the physical and geological aspects of the general planning environment is given. In Chapter 2 the different underlying erosion mechanism at river banks are discussed, followed by a concise description of potential structural countermeasures. An outline of improved and accelerated general planning procedures, including the early identification of priority protection sites, is presented in Chapter 3. Chapters 4 to 6 provide the focal information regarding the actual design of the proposed standardized revetment and groyne structures on basis of the predetermined hydraulic and morphological boundary conditions. The Part II – Manual, contains Chapters 7 and 8, with details on material specifications as well as on construction elements and methods. Finally, Part III provides design plates for structural elements of standardized bank erosion protection structures.

1.2 SCOPE AND APPLICATION OF THE GUIDELINES

First of all, it shall be underlined, that this handbook has not the intention to cover all river related natural and technical aspects, which might be envisaged during the investigation and implementation of bank protection works. Instead, main issues regarding typical structural solutions developed on basis of experiences within a pilot project are considered. These **S**tandardized **P**rotection **S**tructures (SPS) are designed less massive as compared to so-called "hardpoints" to allow for a sustainable use of restricted funds (economic approach). To maintain the function and stability of SPS a sufficient monitoring of the structures after completion is mandatory, to initiate timely repair measures and to ensure the overall integrity of the structures. The fact, that also structures designed at considerably higher safety levels require in general substantial repair costs under such harsh boundary conditions are justifying this decision. Moreover, it has to be taken into account that the high mobility of Bangladesh rivers, resulting in rapid planform changes, is followed by the risk of inadequate and uneconomic solutions in case the completed structure is attacked by the river only for a restricted time (i.e. the service life is small as compared to the structure life time), because the "return period of design conditions" is rather vague during the design process.

For introducing SPS, a categorization is required defining i) the range of expected impact loads and ii), the importance of the protected area. This is indispensable to prevent from overdesign, but also to exclude projects of extraordinary and national importance (Jamuna bridge, protection scheme Dhaka, etc.) from the simplified planning procedures as proposed for SPS. For this purpose four structure categories (SC) were defined, with increasing project relevance from SC1 to SC4. The planning and implementation of SPS is limited to SC2 and SC3 and will be described in this handbook, nevertheless the proposed groyne and revetment structures may also be adapted to advanced requirements, making more comprehensive case investigations obligatory.

For SPS it is suggested, that instead of time consuming physical and numerical model testing, planning data are obtained from field data collection and from more rapid empirical and deterministic bank erosion prediction methods using satellite images or bankline surveys. Despite that, design values are not always present in a measured dataset, but could be derived from modelling after calibration of the model to observed situations. Therefore, regarding the medium-term development of future Standard Master Plans (SMP) it is anticipated, that individual case studies should be partially replaced by a more systematic and methodical approach of generalized boundary conditions. For this purpose, combined efforts of comprehensive field measurements, supported and extended by results from physical and numerical models will provide an optimal data base to further improve the knowledge on the very particular properties of Bangladesh rivers and on the safe design of structural countermeasures against bank erosion.

A simplification and standardization of the planning and implementation process is crucial to allow for timely completion of bank protection structures within a restricted construction window and despite possible sudden changes of the river course.

In this context it has to be stressed, that the performance of SPS is verified for the Jamuna river, at which banks the test structures were situated. The attempt was made to generalize the results to allow for use also for other major rivers of Bangladesh. Although the specific site conditions at the Jamuna may appear to be most critical, it has to be emphasized that in particular meandering rivers might result in much more devastating impacts, e.g. in case the river is attacking the structure almost perpendicular. In addition, some of the generalized coefficients used in various equations for estimation of hydraulic and morphological boundary conditions are subject to further refinement based on future experience and increase of knowledge as compared to the present state-of-the-art. This points at the very significant issue, that all river and design experts involved should contribute to the permanent upgrading process of this handbook to fill remaining gaps of knowledge and to arrive at an optimal river bank management.

Moreover, the discussion on optimal river training techniques is still going on and it will probably take another decade or even generations to find a satisfactory solution. Nevertheless, as mentioned, the proposed SPS are expected to be adaptable to suit the objectives also within different strategies.

1.3 ALLUVIAL RIVERS AND THEIR CHARACTERISTICS

1.3.1 Classification of Alluvial Rivers

Rivers can be divided, according to the topography of the river basin, into the upper reaches in the hills, the middle reaches on the alluvial plain and the lower reaches affected by the sea. Alluvial rivers flow through alluvia, which have been built up by the rivers themselves and which continue to be eroded and deposited by the rivers. They can be classified according to their vertical stability into three types: (a) stable, (b) aggrading and (c) degrading type. The classification depends on the amount and the size of sediment entering the river and its transport capacity for the sediment load. A stable river is in so-called 'regime'. A river section is stable when its transport capacity can cope with the incoming amount of sediment. The regime of a river depends on the magnitude and the variation of its discharge and sediment load, the composition of its bed material, the topography and the composition of its surrounding floodplains and banks. However, the dependence on these boundary conditions is not distinctive, under given certain boundary conditions, several river regimes are possible.

As the sediment load and discharge vary in time, any particular section of the river may be aggrading, degrading and stable at different times. Besides these seasonal changes, an alluvial river normally shows several types from the entry into the alluvial plain towards the estuary. Aggrading and degrading rivers are interim forms, which would eventually develop into a stable river if the boundary conditions remain unchanged. Aggradation is caused by excessive sediment or a decrease in flow velocity, which may be due to a sudden inrush of sediment from a tributary or the widening of a river section. Degradation is caused when sediment intrusion is hindered or when the water level slope increases, for instance downstream from a dam or upstream from a cut-off. Any change in the balance of boundary conditions affects the river regime. If boundary conditions change relatively fast, rivers never reach their regime and remain in a dynamic state of retarded adaptation.

1.3.2 Meandering Rivers

Sinuous rivers or sinuous channels within a river are called 'meanders'. Their planforms change through erosion of the outer banks. The term 'meandering' refers to the process of forming a sinuous course through bank erosion, but is also used for any shift of the banklines (although, e.g., the widening of a river is not a form of meandering) and to distinguish certain river planforms from those of straight and braided rivers (see Section 1.3.3). The terms 'meandering', 'braided' and straight' are not mutually exclusive, because at different water stages the appearance of a river may change, dependent on the number of channels visible at certain water levels.

A quantification of the sinuosity, which is the ratio of the valley slope to the slope of the river (i.e., approximately the length of the actual meandering channel course versus the theoretical shortest passage through a valley), is more precise. Following this definition, the term 'meandering' is used for channels or rivers with a sinuosity larger than 1.25 or 1.5.

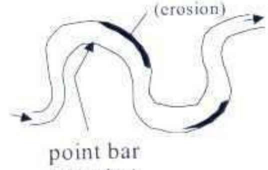


Fig. 1.3-1: Meandering river

1.3.3 Braiding and Anabranching Rivers

The multiple channels of rivers like the Jamuna and the Ganges river are partly braiding and partly anabranching (Fig. 1.3-2). Braiding occurs if the width-to-depth ratio of a river is above a certain threshold, which is the case if the banks of the river are easily eroded. The dominant process of channel shifting is bank erosion, deposition and the dissection of within-channel bars. Discharge variations and aggradation are not obligatory conditions for braiding, nonetheless, rapid discharge variations can promote braiding as they enhance bank erosion, and the number of braids increases with aggradation. Anabranching differs from braiding regarding the fact, that the flow is divided by islands (chars), or sometimes bars, which are large in relation to the channel width.

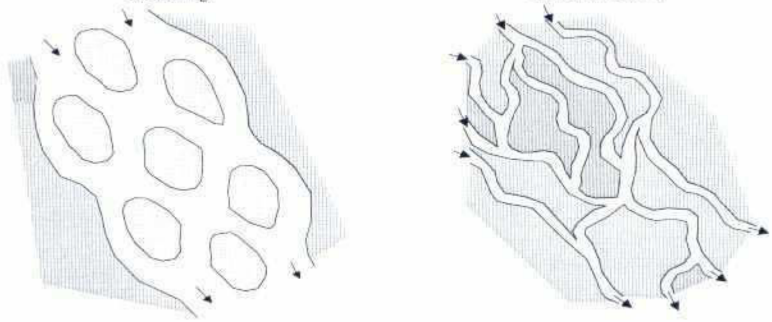


Fig. 1.3-2: Braided and anabranched rivers (schematic)

Each anabranch is a distinct and rather permanent channel with banklines, whereas braids located within the banklines of a single broad channel are shifting more frequently. The dominant process of channel shifting in an anabranched river is avulsion, which occurs if the river captures a new watercourse on the mainland, thus excising an island from the floodplain.

A typical cross-section of a braiding river is shown in Fig. 1.3-3. The active flood plain (AFP) covers the area in which the different river channels may vary. However, these boundaries are subject to change over time and describe more or less the recent area of channel courses. The important parameter of bank full discharge of a river refers to the level of the floodplain. For the major rivers of Bangladesh the statistical return period of the bank full discharge is between one and one and a half years (compare Chapter 4). Chars with a crest level above the bank full discharge are anticipated as quite stable features, whereas lower chars are temporary features only.

Bristow (1987) proposed a classification of river channels into different orders. The entire channel is the first-order channel and comprises a number of smaller second-order channels. The latter have slightly different characteristics and as a result they show a different behaviour in terms of water level slopes as well as the discharge and sediment capacities. The shifting characteristics of the river can be divided according to the order of the channel. The rate of shifting of the first-order channel is 75 to 150 m per year. The second-order channels change their course continuously. Larger channels are abandoned and new ones develop in a few years only. A bank erosion rate of the second-order channels of 250 to 300 m per year is common, but in extreme cases it can be more than 800 m per year.

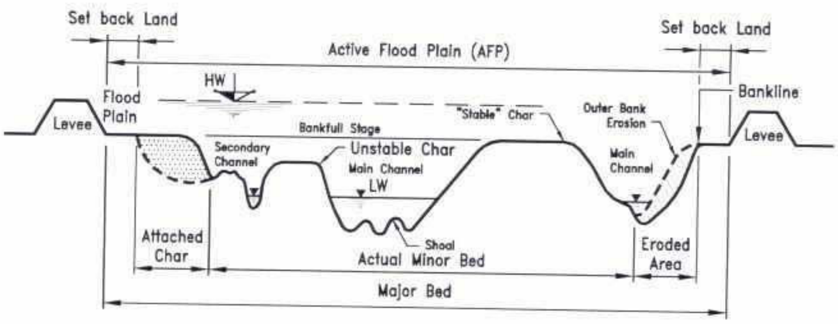


Fig. 1.3-3: Typical cross-section of a braiding river

1.4 RIVERS OF BANGLADESH

1.4.1 River Network and Morphology

The main rivers of Bangladesh are the Ganges and the Brahmaputra originating in the south and north eastern Himalaya respectively as well as the Meghna, draining the Sylhet basin in the north eastern part of Bangladesh. Together with numerous tributaries and distributaries a dense network of rivers in Bangladesh is formed (see Fig. 1.4-1), representing the lowermost alluvial deltaic reach of the fluvial system, which is draining to the Bay of Bengal. One important hydrological aspect of the rivers of Bangladesh is, that the rise and fall of the river stages are only very weakly dependent on the local rainfall, because 92 % of the catchment lies outside Bangladesh. The rivers have changed their courses frequently in the past, the Brahmaputra switched from its eastward course in favour of a southward course along its small former distributary, the Jamuna river, and the old Brahmaputra course became a small distributary of the Brahmaputra/ Jamuna river itself. Previously, the Ganges used to empty into the Bay through the Arial Khan river, but after capture of the Brahmaputra/ Jamuna river flow it shifted north-eastward joining the Upper Meghna.

The **Brahmaputra** (called **Jamuna** in Bangladesh) has one major tributary on the right bank, the Teesta, and two left bank tributaries, the Old Brahmaputra and the Dhaleswari river. It is a wandering braided river with an average bankfull width of approx. 11 km.

The **Ganges** is a widely meandering river with a bankfull width of about 5 km, with one major tributary on the left bank, named the Mahananda. The right bank distributary, the Gorai, carries part of the high stage flow of the Ganges to the Bay of Bengal. The river Ganges rises from the southern flanks of the Himalayas in India. Before meeting with the Brahmaputra/Jamuna in Bangladesh, the river stretches over 2,200 km, draining an area of about 1,000,000 km^2. After changing its course about 500 years ago, the river is now in a dynamic equilibrium. The sinuosity of the river is decreasing. It is behaving as a wandering river, in particular the part downstream from Hardinge Bridge, changing its planform between meandering and braiding. An active corridor of the Ganges has been identified, within which the risk of bank erosion is high, but also some embayments and nodal points along the river have been observed, in between which the river wanders. The erosion rate of the Ganges is quite high with almost similar values as for the Brahmaputra/ Jamuna river. However, the erosion rate is considerably reduced when the river attacks the highly erosion resistant boundary of the corridor.

Geo-morphologically, the river **Padma** is still a young river. It is now in a dynamic equilibrium. A stretch of about 90 km is almost straight and the river planform is a combination of the meandering and braiding type indicating a wandering river swinging within an active corridor. The boundary of this corridor is often attacked by the river resulting possibly in widening of the corridor. The variation of the total width of the river is quite high ranging from 3.5 km to 15 km. The braiding intensity of the Padma is low and typically there are only two parallel channels in the braided reach. The shifting processes of the channels are quite rapid.

Summarizing, it can be stated that the bank erosion rates of the three main rivers are very similar. However, at Ganges and Padma, the bank erosion is restricted to the boundary of the active corridor, which consists of alluvial and deltaic silt deposits, whereas the floodplain outside of it is more resistant to erosion. At the Brahmaputra/ Jamuna the flow attacks any of the banks and new channel courses outside the active flood plain are created frequently.

The **Upper Meghna**, which originates in the Shillong Plateau and foothills, is a relatively small river having a bankfull width of about 1 km only. The Ganges and the Jamuna meet near Aricha forming the Padma river, which flows south-eastward until it reaches the Upper Meghna near Chandpur. This reach of some 120 km is relatively straight with one important right bank distributary, the Arial Khan river, which carries part of the Padma flow directly into the Bay of Bengal.

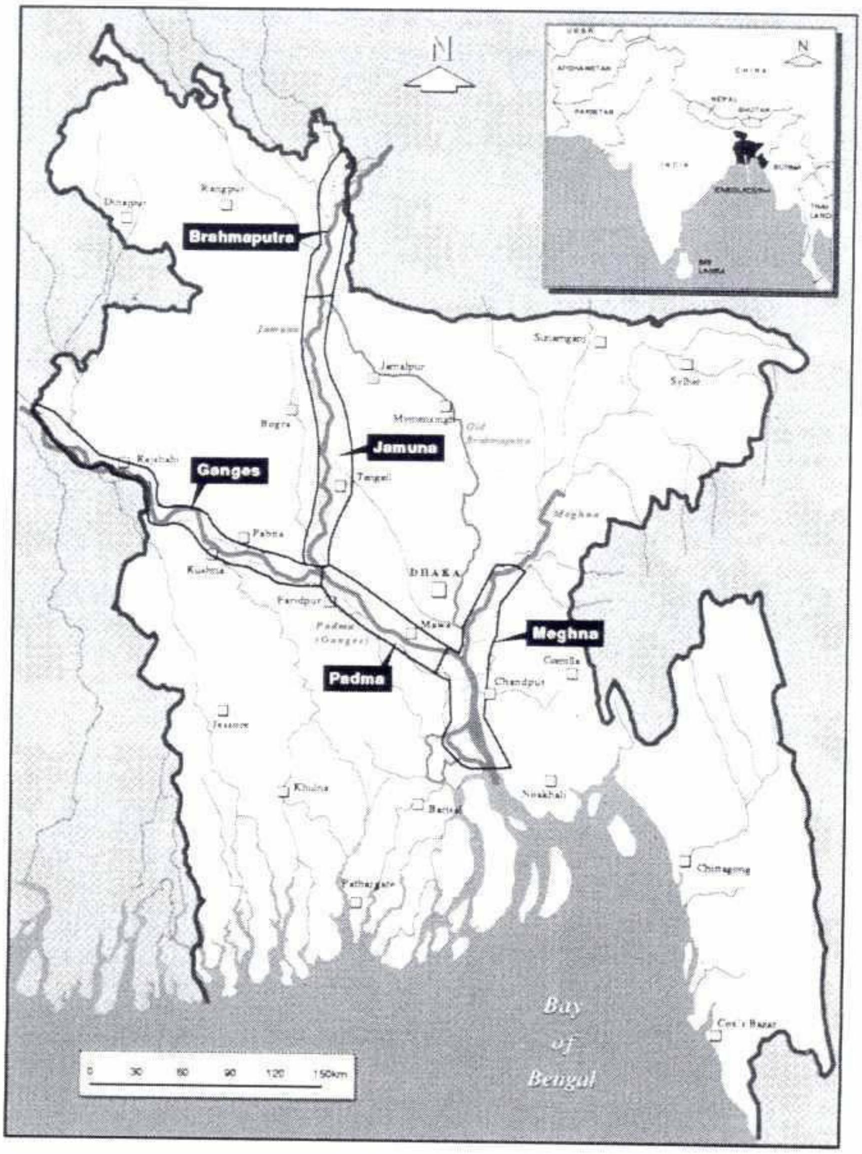


Fig. 1.4-1: Major rivers of Bangladesh

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1.4.2 Hydrological Aspects

The hydrology of the rivers of Bangladesh is governed by the surface and subsurface runoff generated by the rainfall distribution in the respective catch areas. In the winter (October until March), north-eastern monsoons occur due to the presence of a high pressure zone in the Asian highlands. Therefore, Bangladesh experiences relatively moisture free continental winds, whereas during summer a low pressure zone develops in Central Asia generating moisture-laden winds from the Arabian Sea and the Bay of Bengal, resulting in a distinct seasonal rainfall distribution. In January/February there is only light rainfall of about 25 mm. During the period from March to May early summer thunderstorms, known as Northwesterlies, occur with a rainfall of about 90 mm in the northwest and about 420 mm in the northeast. From June to September the southwest monsoon occurs with heavy rains.

The annual average rainfall in the Brahmaputra/ Jamuna area is some 1,900 mm and the water yield per km^2 of the drainage basin is about 0.03 m^3/s. The Brahmaputra/ Jamuna drains an estimated volume of $620 \cdot 10^9$ m^3 of water per year into the Bay of Bengal with an annual average discharge of 19,600 m^3/s. Each year the river reaches a bankfull discharge at about 48,000 m^3/s, with a maximum of about 100,000 m^3/s during the 1988 flood. The average water surface slope is approx. 7 cm per km ($7 \cdot 10^{-5}$), decreasing in downstream direction from $8.5 \cdot 10^{-5}$ at the upstream end within Bangladesh and $6.5 \cdot 10^{-5}$ near the confluence with the Ganges. Dependant on the rainfall distribution, the river stage varies by about 6 m. Regarding the hydrograph of the Brahmaputra/ Jamuna, seven phases can be distinguished (see Table 1.4-1 for the location Bahadurabad). The most important characteristic of the hydrograph is the existence of a long-lasting peak between July and September.

Phases	Months	Variability Characteristics
1	March to May	Slow rising
2	June	rapid rising
3	July and August	varying peaks
4	September	very slow falling
5	October	Rapid falling
6	November and December	Slow falling
7	January and February	Nearly constant

Table 1.4-1: Characteristic phases of the annual hydrograph of the Jamuna river

The river **Ganges** has the lowest water yield per km^2 among the main rivers of Bangladesh. It is about 0.01 m^3/s with an annual rainfall of some 1,200 mm. The river drains $252 \cdot 10^9$ m^3 of water annually to the Bay of Bengal. The annual average discharge of the Ganges is only about 56 % of the Brahmaputra/Jamuna, varying from a minimum of 1,000 m^3/s to a maximum of 70,000 m^3/s over the year. A seasonal water level variation of about 8 m can be observed at Hardinge Bridge. The water surface slope of the river is $5 \cdot 10^{-5}$, i.e. 5 cm per km.

The **Padma**, which is draining the combined flow of the Brahmaputra/Jamuna and the Ganges and the lower reach of which is weakly influenced by the tide during the period from December to April, has an annual discharge of about 28,000 m^3/s. The seasonal water level variation is about 6 m.

The **Upper Meghna** is representing one of the areas of highest rainfall in the world (annual rainfall of approx. 4,900 mm), the water yield is about 0.06 m^3/s per km^2. The annual average discharge is about 4,800 m^3/s and the river drains some $151 \cdot 10^9$ m^3 of water per year into the Bay of Bengal. The lower reach of the river becomes tidal during the period from December to April. The seasonal water level variation is about 5 m.

The **Lower Meghna** is a tidal reach, which carries almost the entire fluvial discharge of the Brahmaputra/Jamuna, the Ganges and the Upper Meghna. The net discharge varies from 10,000 m^3/s in dry season to 160,000 m^3/s in monsoon season.

River characteristics		Major rivers Jamuna	Ganges	Padma	Upper Meghna	Dhales-wari	Gorai	Arial Khan	Old Brahma-putra
Catchment area (million km^2)		0.57	1.09	-	0.077	-	-	-	-
Gradient (m/km)		0.07	0.05	0.04	0 to 0.02	0.045	0.04	-	0.07
Average discharge (m^3/s)		19 600	11 000	28 000	4 800	600	1 400	2 600	500
Bankfull discharge (m^3/s)		48 000	43 000	75 000	-	-	-	-	-
Return period of bank-full discharge (year)		1.00	1.40	1.05	-	-	-	-	-
Width (m)		11 000	5 000	7 000	1 000	256	-	-	330
Average depth (m)		5.0	4.5	-	4.7	9.0	-	-	7.0
Seasonal water level variations (m)		6	8	6	5	-	8	-	-
Tidal influence		none	none	in winter	in winter	lower reach in winter	lower reaches	entire reach	none
Median bed-material grain size (µm)	overall	190	120	90		150	179	180	180
	active bed	200	140	-		-	-	-	-
	low-velocity areas	35	30	-		-	-	-	-
Vertical stability		stable	stable	stable	-	-	aggrad-ing	degrad-ing	aggrad-ing
Meander wave length (km)		-	29	-	9.0	-	9.2	4.2	5.5
Channel sinuosity (-)		-	1.2	1.1	-	1.5	1.7	1.8	1.4
Brice braiding index (-)		4 - 6	-	-		-	-	-	-
Regime $B = aQ^b$	A	8.97	9.97	4.76		-	-	-	-
	B	0.57	0.555	0.62		-	-	-	-
Regime $H = aQ^b$	A	0.4	0.28	0.28		-	-	-	-
	B	0.26	0.29	0.30		-	-	-	-
Degree of non-linearity in relation between sediment transport and flow velo-city		3.66	5.0	-		-	-	-	-
Regime relations for individual channels: B = width (m), h = water depth (m), Q = bankfull discharge (m^3/s)									

Table 1.4-2: Important characteristics of major rivers of Bangladesh

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1.4.3 Geological and Sedimentological Aspects

Geologically Bangladesh can be divided into three broad physiographic regions. These are the Tertiary Hills, the Pleistocene Uplands and the Recent Plains, which are also the major relief features of the country. The Recent Plain can be further subdivided into Piedmont Alluvium, Flood Plains and the Tidal and Estuarine Flood Plains.

The country consists primarily of deltaic alluvial sediments of the three great rivers Ganges-Padma, Brahmaputra/ Jamuna, Meghna and their tributaries. The basins of the Brahmaputra/ Jamuna and the Ganges are bounded to the tectonically highly active Himalayas mountain ranges, which are subject to severe erosion contributing to the heavy sediment load in the Ganges and the Brahmaputra/ Jamuna. The entire country of Bangladesh is a part of the Bengal basin, filled in the tertiary-quaternary geological period. The basin is an area of subsidence, which is balanced by the deposition of sediments supplied by its river system. The thickness of the sediment cover above the basement rock increases from about 180 m along the Rangpur-Dinajpur axis to over 18,000 m in the south eastern part of the country.

Pleistocene sediments constitute the floodplain deposits of the earlier Ganges and Brahmaputra river. They are elevated relative to the recent floodplain and can be identified by their tone and texture. They are reddish in colour, highly oxidised, compacted and contain less water than the recent sediments. The latter are typically dark, loosely compacted and with high water contents and appreciable quantities of organic components as shown in Fig. 1.4-2.

Fig. 1.4-2: Soil stratification with more recent sediment layers (organic and darker in colour) on top of Pleistocene sediments

The rivers of Bangladesh are characterised by a fine sedimentary environment. The consequence is that the threshold velocity for sediment mobility is low, about 0.2 m/s. Hence, the

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rivers are highly mobile with continuous reworking and deformation of their beds and banks transporting huge quantities of sediment.

The floodplain of the three main rivers Brahmaputra/ Jamuna, Ganges and Padma consist of recently deposited sediments. The oscillation zone of these rivers consists of alluvial sand and is covered by alluvial silt or deltaic silt. The latter are expected to be cohesive to some extent and exert a higher resistance against erosion than alluvial sand. The extent of cohesion, however, depends on the clay and mineral contents as well as on the age of the deposition. Such minerals are carbonate minerals from the limestone of the Himalayas and the Pleistocene soil in northern India. Carbonate minerals are present in the Ganges floodplain, but absent in the floodplain of the Brahmaputra/ Jamuna. Hence, the resistance to erosion is expected to be higher at the Ganges than along the Brahmaputra/ Jamuna.

In general, alluvial sediments may range from fine silt to gravel, whereas a large part (about 60 to 85 % of the total volume) of the sediment load in all rivers of Bangladesh consists of silty materials. The recent sediments near the present courses of the Brahmaputra/ Jamuna, the Ganges and the Padma can physically be classified as follows:

- Alluvial sand
  Coarse to fine silty sand of light to brownish grey colour constitutes the channel bars and the levee deposits along the rivers and the large tributaries. The Jamuna sand ranges in size from coarse to fine, while the Padma river sand is medium to fine. The boundaries of the alluvial sand represent probably the oscillating boundaries of the rivers.
- Alluvial silt
  Light to medium grey, fine sandy to clayey silt, mainly deposited in the flood basins and in interstream areas. It is normally poorly stratified and the average grain size decreases with the distance from the main channels. These silt deposits surround the alluvial sand strips of the Brahmaputra/ Jamuna and are only found at the left banks of the Ganges and Padma.
- Alluvial silt and clay
  These sediments of medium to dark-grey colour are located close to the boundary of the active corridor of the Brahmaputra/ Jamuna for a few kilometres at the left bank and at a few stretches of the left banks of the Ganges and Padma.
- Marsh and clay peat
  These deposits of grey clay, black peat and yellowish-grey silt are found in subsiding areas near the left bank of the Ganges and Padma.
- Deltaic sand
  This light to yellowish grey, fine to silty sand was mostly deposited during floods in channels and floodplains including channel bars and point bars. They are found on the right bank of the Ganges and Padma.
- Deltaic silt
  This grey, fine sandy silt to clayey silt were deposited in flood basins and are found almost all over the right bank of the Ganges and Padma. At the upstream reach of the

Ganges the deposition is older than the deposition at the downstream part of the Padma. The cohesiveness of the material depends on the age of the deposition.

The **Brahmaputra/ Jamuna** shows the largest sediment grain sizes and transports the largest sediment load. The median diameter of the bed-material decreases from 220 μm near Chilmari to 165 μm near Aricha. The mean annual suspended sediment transport is estimated to vary from 390 to 650 million tons.

The river **Ganges** has a more fine-sedimentary environment than the Brahmaputra/ Jamuna. The median bed-material diameter is about 120 μm and the mean annual suspended sediment transport varies from approx. 200 to 550 million tons.

The bed-material sediment of the **Padma** varies from 140 μm in the upper reaches to 90 μm in the lower reaches. Due to the combined flow of the Brahmaputra/ Jamuna and the Ganges the estimated mean annual suspended sediment transport varies from 600 to 1200 million tons.

The bed-material of the **Upper Meghna** has a median diameter of 140 μm and the river transports some 15 million tons of suspended sediment annually. Among the distributaries, the **Dhaleswari** river transports approx. 20 million tons and the **Gorai** river some 50 million tons annually.

1.4.4 Seismic Aspects

Bangladesh is located in an active tectonic region related to the convergence and collision of the Eurasian and Indian Plates. The results of such tectonic activity are the existence of several deep-seated faults, structural upwarping or downwarping and episodic earthquakes.

In the 19th and 20th century over 200 major earthquakes occurred in and around Bangladesh, but there seems to be no seismically active fault in the territory. However, causative faults and regions of high seismic activity exist in neighbouring India and Burma in the north and east of Bangladesh. Earthquakes in these areas can affect the adjacent regions in Bangladesh as well. Bangladesh can be sub-divided into three seismic zones, as shown in Fig. 1.4-3, with increasing severity from Zone 1 to Zone 3.

The intensity of the earthquakes to be expected in the various areas is generally expressed by the magnitude of the horizontal seismic acceleration. A possible simultaneously vertical acceleration is generally negligible low, compared with the acceleration due to gravity.

If a structure is to be built in an earthquake region, the effect of possible earthquakes at the site area must be taken into consideration. This holds in particular for river training and bank protection structures, since the horizontal seismic acceleration, which occurs during a quake affects the active and passive earth pressure, the safety against foundation failure, slope failure, sliding and also the shear strength of the soil. Under unfavourable circumstances the latter may temporarily disappear completely.

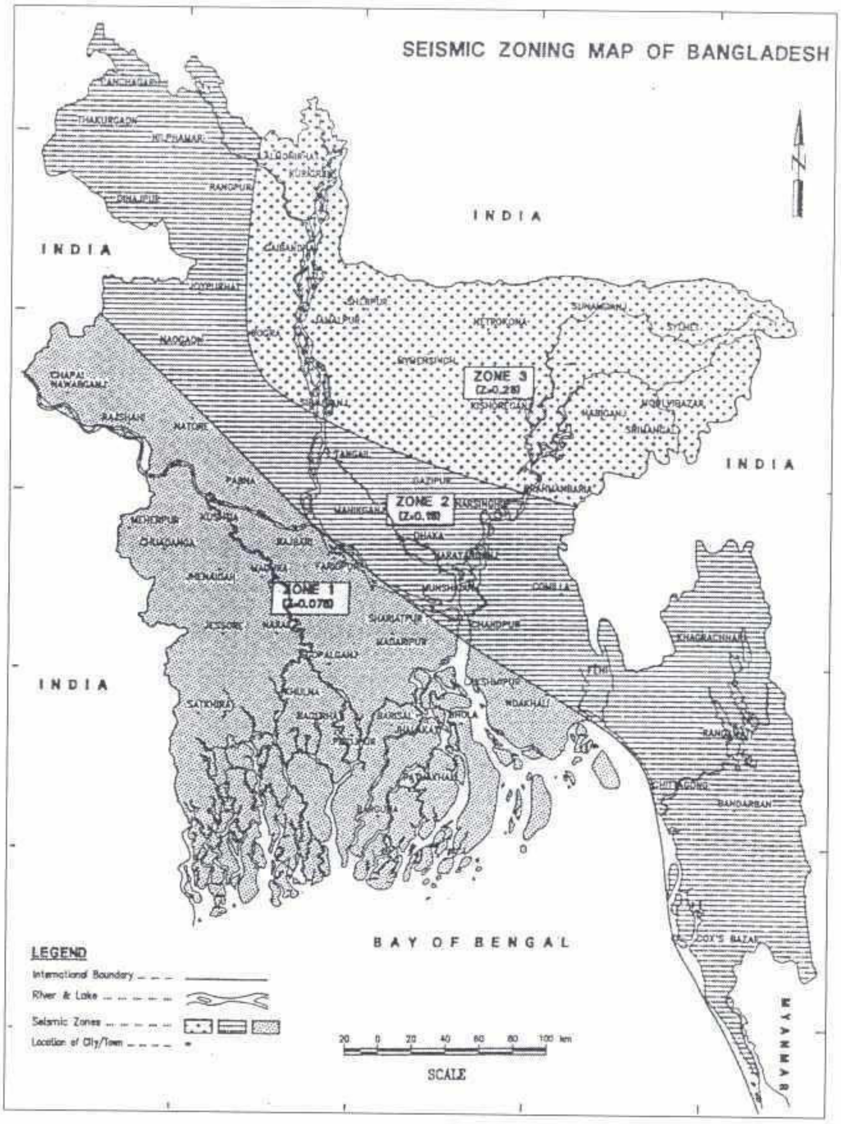


Fig. 1.4-3: Seismic zoning map of Bangladesh (adopted from: Guide to Planning and Design of River Training and Bank Protection works, BWDB)

2 BANKLINE EROSION AND POSSIBLE PROTECTION MEASURES

2.1 PROCESSES OF EROSION AND MASS FAILURE OF BANKS

2.1.1 Introduction

The stability of unprotected river banks depends on a number of factors which have to be assessed carefully in the process of selection and design of suitable protection measures. A reliable assessment of potential causes of bank failure is indispensable for the success of any measures, i.e. for the integrity of the selected bank protection system and thus the stability of the river bank. However, it is stressed that the proposed protection scheme can possibly affect the overall stability of the river or a river channel.

Bank erosion can occur in stable as well as in unstable rivers or river channels. Although the morphology of stable rivers is in a state of equilibrium with regard to the governing physical processes and is not expected to change significantly the general shape and dimensions, some local erosion and deposition is likely to occur, especially in meander bends.

Modification of flow velocity, discharge, sediment load and river morphology in unstable rivers/ river channels are major factors initiating erosion and deposition. How fast the river responds to changes in boundary conditions depends on the natural stability of the subsoil and the extent of changes. Successive erosion and deposition often leads to rapid changes in the river planform and slope. Increased meandering reduces the channel slope, whereas straightening of the channel through cut-offs in most cases increases the local gradient. Any changes in bed elevation can also promote rapid bank erosion. Incision can destabilise a bank. Strong accumulation of sediments, followed by development of bars and islands promotes rapid widening and development of braided channels.

2.1.2 Surface Erosion of Banks

In general surface erosion of river banks or along the river bed occurs if the driving erosive forces are exceeding the resistive forces of the individual grains or of the conglomerates in case of cohesive materials. The main impacts responsible for surface erosion at river banks are (see also Fig. 2.1-1):

- current induced shear stress
- wave loads (wind-generated waves; ship- and boat-generated waves)
- seepage (excessive pore pressure)
- surface runoff
- mechanical action (desiccation, ship impact, activities of humans and animals)

Shear stress induced by current flow is the main hydraulic erosion factor. Although the primary current (velocity component in direction of the river course) is much larger, in river bends secondary currents, which are mainly generated by inertia forces, are the key factor for the asymmetric cross sectional profiles.

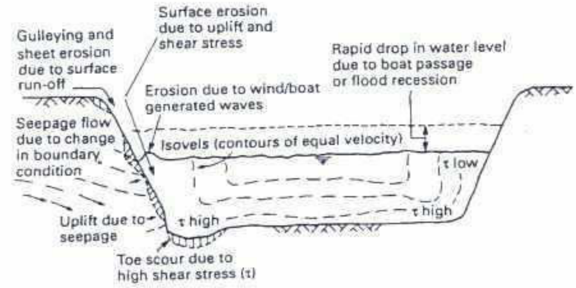


Fig. 2.1-1: Processes of surface erosion (adopted Hemphill and Bramley, 1989)

However, due to the velocity gradient in the primary current also in straight river sections secondary flows exist (Fig. 2.1-2). Moreover, in turbulent flows three-dimensionality of the current distribution is amplified by irregularities in the cross-sectional shape of the channel or by changes in the roughness of the river bed. In most formulae developed for computation of bed scour the secondary flow component is not considered explicitly, but is taken into account through empirical coefficients.

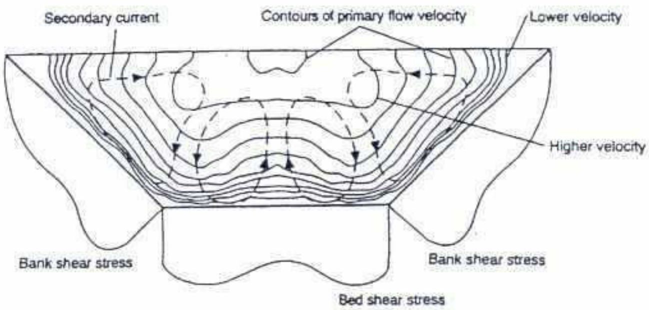


Fig. 2.1-2: Contours of primary and secondary flows as well as the shear stress distribution in a trapezoidal channel (adopted Hemphill and Bramley, 1989)

The geo-morphological processes in natural rivers are even more complex. Obstructions to the flow as well as variations of the roughness of the river bank and/ or river bed cause changes in the velocity distribution and the secondary flow patterns. As a consequence the river starts to develop bends. Once the meandering process has started, it tends to sustain itself. This development is also possible in lower river reaches, where the slopes are more gentle, in particular in alluvial soils. Significant variations of the shear stress along straight and meandering sections can be observed (see Fig. 2.1-3). Scouring occurs at the outer bank of a meander bend during flood flows, since the shear stress (i.e. the velocity) adjacent to the bank increases in the direction of the flow. The eroded material is transported as bed load and suspended load and will be deposited in areas of smaller flow velocities. Consequently, deposition of sediment occurs at the inner bank along a river bend, caused by the decrease in shear stress in downstream direction.

In meandering and braiding rivers with outflanking channels, erosion generally occurs at concave banks rather than at convex banks. Maximum values of the bank shear stress during high water stages (bank full discharge) are observed at the base of the outer bank in the meander bend. At low flows, peak values occur at the meander apex. Parallel with increasing discharge, the peak values move downstream towards the inflection point. For that reason the actual location and rate of bank erosion depends on the residence time of prevailing boundary conditions, i.e. the water level stages and associated flow conditions and shear stresses.

The influence of wave action is mainly important along the actual bank line, i.e. the transition between water and adjacent flood plain of a river. In this zone the wave energy is transformed to wave run-up and wave run-down, creating high velocity components normal to the river slope, thus increasing shear stress and erosion. In addition, dependent on the breaker type - especially under plunging breakers, extremely high pressure heads may occur, which have considerable destructive force when hitting the bank line.

Seepage or surface runoff are typical sources of mass failures, which are described in the following Section 2.2.

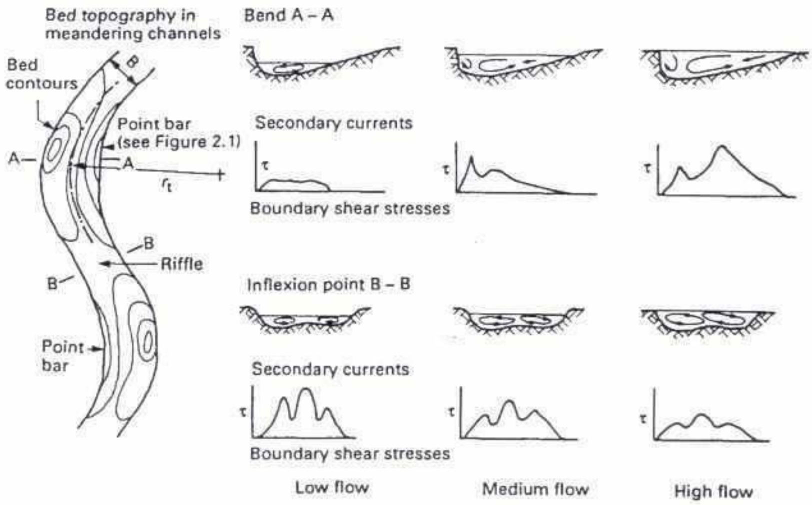


Fig. 2.1-3: Secondary currents and boundary shear stress in a meandering channel (after Hey, 1986, in Hemphill and Bramley, 1989)

2.1.3 Mass Failure of Banks

Mass failure of river banks can be divided into block failures and slip failures, initiated by different processes, which are illustrated in Fig. 2.1-4. The actual failing of a river bank may not follow immediately after an impact, in some cases the failure process takes several days. On the other hand this indicates that a failure may occur without warning at almost any time, if active surface erosion, toe scour is prevalent or an additional load is applied to the bank. The risk of mass failures is increased during heavy rain and during quick fall after high river stages. Potential failure modes of slip and block failures are listed below:

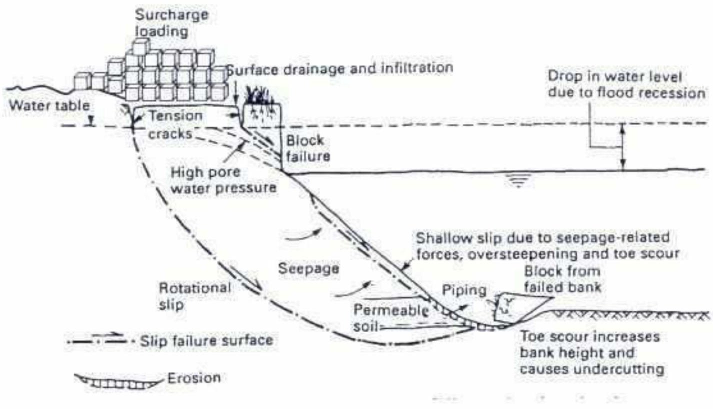


Fig. 2.1-4: Processes responsible for mass failure of a river bank

<u>Slip failures</u>

- In case of non-cohesive material and a shallow bank angle the failure surface is usually approximately parallel to the slope angle. Water seepage can substantially reduce the stability of the bank, whereas vegetation will normally help to stabilise against failure.
- Steep or almost vertical banks of non-cohesive material can fail along a plane or slightly curved surface. This is often the case when the river water level is low relative to the total bank height.
- If relatively deep tension cracks have developed on the surface of the river bank, failure occurs by sliding and/or toppling. This failure mode is little affected by the groundwater table, but is more likely if the crack fills with water.
- Deep-seated rotational failure is possible in cohesive soil where the banks are steep and moderately high. If the soil is relatively homogeneous, the failure surface may follow a circular arc.
- It is also possible that layers of weak material affect the actual shape of the failure surface, which may then include logarithmic spirals or even planar sections. Both types of failure can extend beyond the toe of the bank. The stability is significantly affected by the position of the water table and if the tension cracks are filled with water.

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

Block failure

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

Typical modes of river bank failure are exemplary shown for slip failures and for block failures in Fig. 2.1-5 (adapted from Hemphill and Bramley, 1989).

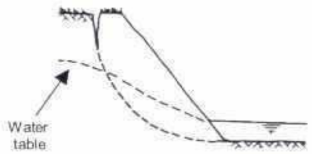
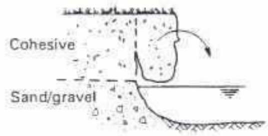
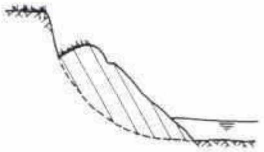
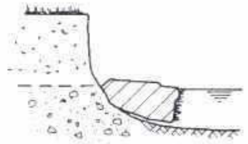
	Slip Failure	**Block Failure**
Before		
After		
	Rotational failure in homogeneous material • usually on moderately high or steep banks • usually in cohesive material • tension cracks reduce stability particularly when water-filled • significantly affected by position of water table • failure may extend beyond toe	Failure of composite bank • failure with upper soil in tension, followed by rotation • after failure block usually remains intact with vegetation towards river • failure can also be by shear

Fig. 2.1-5: Typical modes of river bank failure (adopted Hemphill and Bramley, 1989)

2.2 GENERAL CONCEPTS OF BANK PROTECTION MEASURES

2.2.1 Introduction

In order to prevent erosion of river banks, suitable counter measures are required. These may be single or combined structural and non-structural measures. In general, three relevant concepts of erosion counter measures are existent:

- River training measures, which are intended to influence the flow conditions or channel properties downstream of the man-made intervention (active measures)
- Structures, which are aimed to decrease the hydraulic impacts directly in front of an area to be protected (partly active and passive measures), e.g. groynes
- Structures to protect the actual bankline without relevant active interference on the fluid (passive measures), i.e. revetments

Either of them must be designed properly to resist hydraulic loads and to prevent the river channels from uncontrolled changing. These guidelines target on suitable tools for planning and implementation of permanent structural active and passive bank protection measures, e.g. groynes and revetments, whereas river training measures, as improved low-cost measures within an active flood plain management, will be discussed only briefly in Section 2.2.2.

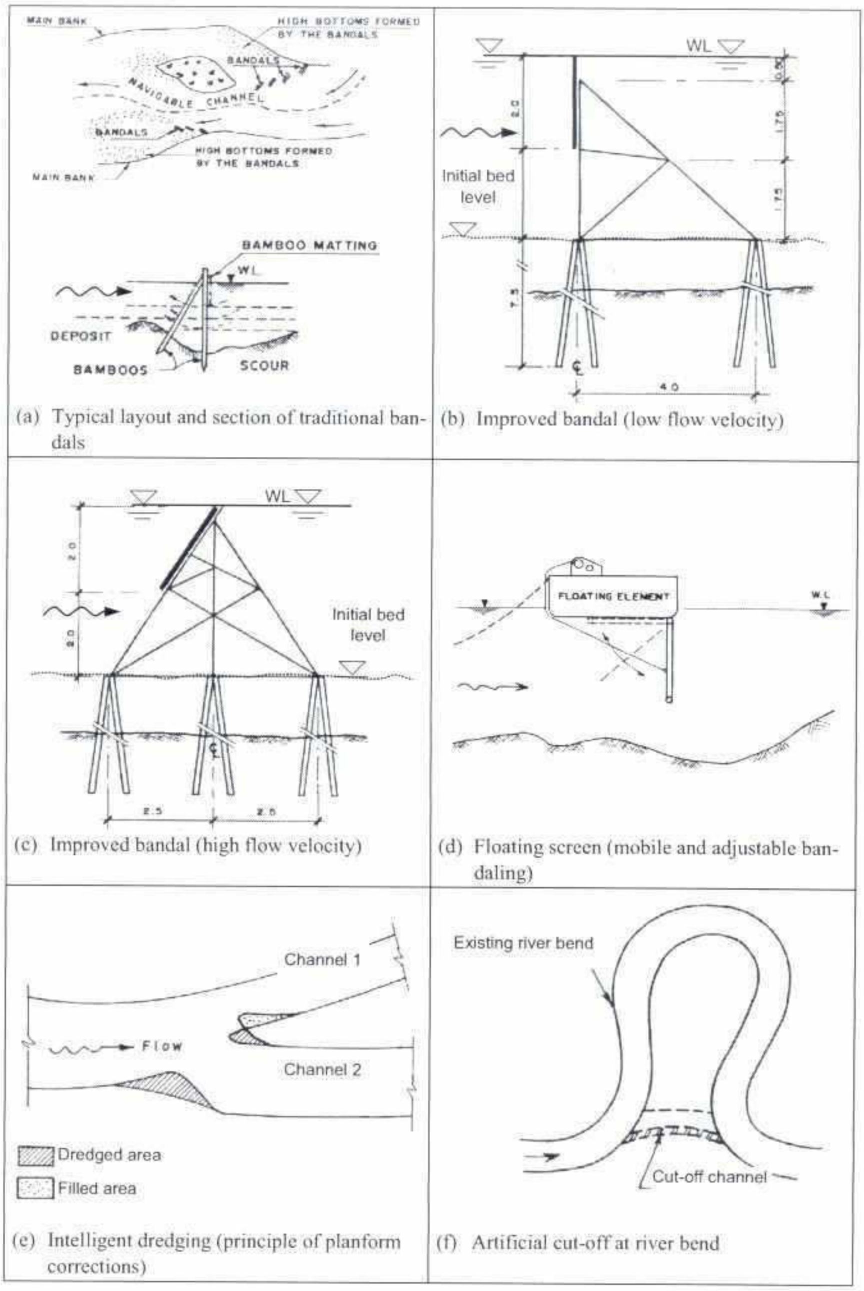
2.2.2 River Training Measures

River training measures comprise permanent and recurrent measures, which are built either on the mainland, the flood plain, attached chars or which are built as floating structures. A large variety of traditional and low cost measures is existent, such as bandals, bamboo groynes (spurs), porcupines, sills, floating screens, cut-offs, so-called intelligent dredging schemes, etc. that are being used in Bangladesh and surrounding countries. In general, due to constraints of the natural material and hand driven equipment used, these measures are restricted to a certain range of applications and comparatively low hydraulic impacts. Some improvements have been introduced to allow for use also under moderate boundary conditions. Which of the above measures is selected, depends mainly on the strategy followed and on the prevailing or expected hydraulic loads as well as on available materials. Some brief details on the most relevant methods are given in the following.

2.2.2.1 Bandals

"Bandals" are vertical vanes with a gap near the river bottom (see Fig. 2.2-1 (a)), which are in general temporary placed during descending water level stages at one or at both sides of a channel. Bandals consists of a framework of bamboo driven into the riverbed and supported by struts. Bamboo mats are fixed to the framework near the water level. Bandal structures are commonly applied to improve/ maintain the depth of river channels for the navigation during low water periods or to close secondary channels by redistribution of discharge and sediment load at bifurcations. The individual bandals are oriented at an angle of 30 to 40 degrees (inclined downstream) to reduce the flow velocity near the bankline and to increase the flow velocity in the thalweg, thus, to narrow and deepen the main flow cross-section of discharge. As a rule of thumb the blockage of the flow section should be about 50% (i.e. panel height and clearance from lower panel edge to river bed level at time of installation should be about equal.

Due to the fact, that the efficiency of bandalling depends strongly on the actual water depth, which is varying quite intensively, a good performance is achieved only during particular water level stages, where an optimal blockage ratio of the individual bandals is achieved.



(a) Typical layout and section of traditional bandals

(b) Improved bandal (low flow velocity)

(c) Improved bandal (high flow velocity)

(d) Floating screen (mobile and adjustable bandaling)

(e) Intelligent dredging (principle of planform corrections)

(f) Artificial cut-off at river bend

Fig. 2.2-1: Typical river training measures

Furthermore, the prevailing sediments must allow for a certain mobility, to allow for re-shaping of the river bed.

In order to maintain the efficiency of bandal structures over a longer period during the low water phase, improved bandals with increased stability and adjustable mats have been developed to cope with the fluctuations of the water level (see Fig. 2.2-1 (b), (c)). These structures are used for water depth up to 4 m to 5m, therefore a proper foundation by use of timber piles or bamboo bundles of suitable length is required in order to bear stronger hydraulic loads.

2.2.2.2 Floating Screens

To further increase the efficiency and flexibility of bandals floating screens have been developed. These consist of a floating element e.g. a pontoon or barge to which a vertical screen is fixed (see Fig. 2.2-1 (d)). The screens are adjustable in depth to maintain optimal blockage. A number of floating screens can be linked together in a chain, which can be placed at a suitable location and direction in the river fixed by an appropriate anchor system.

2.2.2.3 Dredging Measures

Dredging is an essential element of river training works for river sections where an adjustment of the flow cross-section and the river bed profile is planned. This can be achieved either by substantial dredging, deepening one channel while filling the other or by changing the flow conditions at the bifurcation through correction of the planform or bed profile resulting in erosion of the flow attracting channel (see Fig. 2.2-1 (e)). Artificial cut-offs and closure dams are specific types of dredging measures.

Dredging can also affect the flow and the shape of the cross-section in an eroding outer channel, where the thalweg is normally located at the outer bank (see Fig. 2.2-2). The reshaping will influence the horizontal flow distribution, nevertheless after completion of the dredging works, the spiral flow (secondary components) will continue eroding the channel bed near the outer bank unless the bed is suitably protected. The eroded material at outer banks can also be refilled by supply of dredged material covering the lower part of the endangered bank slopes (see Fig. 2.2-3).

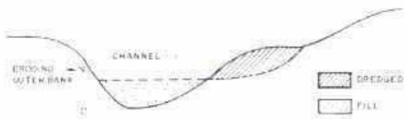


Fig. 2.2-2: Changing of a cross-section

Fig. 2.2-3: Protection of outer bank by dredged material

The comparatively large volume of material to be dredged requiring substantial dredge capacity, restrict applications to secondary or smaller channels and to low water stages. If the river-bed is not protected after re-shaping, dredging measures are rather unstable and require recurrent activities .

2.2.2.4 Artificial Cut-off

Cut-offs may be initiated to reduce the erosion attack at well developed concave bends (see Fig. 2.2-1 (f)) by excavating a channel in full dimensions or as a pilot channel which is eroded to the final extend during rising water level stages, provided that the existent sediment is fairly mobile sandy soil. The rate of flow directed in to the pilot channel should be at least 25% to 30% of the bankfull discharge. The creation of a cut-off results in an increase in water slope, bed slope and flow velocity, in parallel the water depth will decrease, which usually lead to local erosion. The eroded material is normally deposited in the reach downstream from the cut-off.

2.2.2.5 Closure Dams

Secondary channels in a braided river or river bends may be also closed by earthdams, which are built during low water stages. Typical locations are at bifurcations or at narrow areas of a channel. The slopes and the crest of the dam must be protected against wave and current attack, in particular in case overtopping is allowed. The toe of the slopes must be protected by suitable aprons and special attention must be also given to the transition between earth dam and banks of the channel. Bypassing of the closure dam by new channels eroding the transition zone must be avoided.

2.2.3 Groyne Structures

Groyne structures can be considered as partly active and passive measures. The decisive criterion in this regard is the structure permeability which will be discussed below. Groynes are built perpendicular or at a certain angle to a riverbank, protruding into the river. The main objective is to deflect the flow away from critical banks, i.e. for controlling erosion, to establish and maintain safe navigation channels as well as to reduce the flow velocity downstream from the structure to initiate siltation in this area.

There is a large variety of groynes with regard to design, hydraulic properties as well as to construction materials and methods. They can be classified based on the

- inclination and shape
- permeability and submergence

of the individual groynes.
Groynes inclined in upstream direction are called repelling groynes, because of their ability to divert the flow away from the structure (Fig. 2.2-4). In contrast, attracting groynes point downstream and attract the flow towards the structure's head and thus to the river bank (Fig. 2.2-5). Therefore, this type of groyne should be placed at the inner bend of a river course to protect the outer concave bend. As demonstrated in the Figures single groynes provide only local protection. For that reason, normally several groynes are combined to form a groyne field to increase the efficiency and to enlarge the stretch of protected river bank.

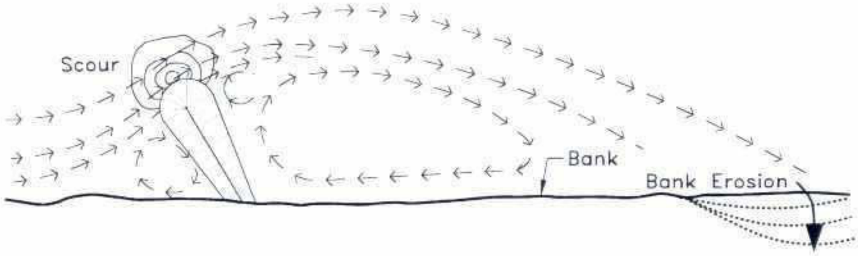


Fig. 2.2-4: Single repelling groyne

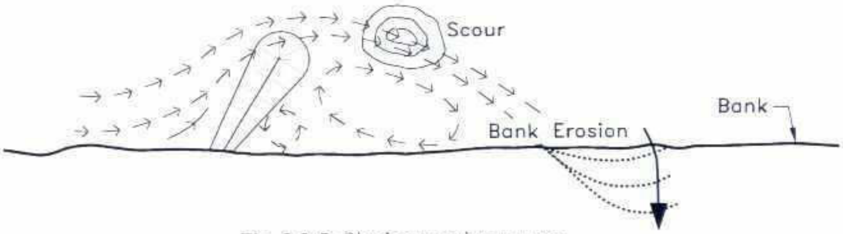


Fig. 2.2-5: Single attracting groyne

Due to the fact, that groynes act like a blockage to the river flow, the flow lines will merge in front of the groyne head resulting in high local velocities and scour. To reduce this effect, which might destabilize the groyne structure and normally requires massive scour protection, and to further improve the performance (i.e. the protection capability), a large number of differently shaped groyne heads have been tested over the last decades. Some alternative groyne head designs are given in Fig. 2.2-6.

Straight groynes (Fig. 2.2-6 (a)) without any extra head protection as compared to the trunk are most unfavourable regarding the head stability. To improve the scour resistance a more extended round head (b), a so-called molehead, may be employed, which provides extra volume of scour protection material and a gentler transition between groyne head and river bed.

T-head (c) or L-head (d) groynes are introduced to give additional guidance to the flow, to improve the bank protection, to reduce the scouring at the groyne head and to increase sediment depositions downstream from the groyne. T-head and L-head groynes generally need strong cover layers.

Groynes with curved trunks are known as hockey shaped groynes (e). In particular if the groyne is curved in flow direction (inverted hockey shaped groyne), this type allows a reduction of the scour material required as compared to (c) and (d) if a strong attack at the head is expected.

In addition, many combinations and specific designs of groyne heads are existent.

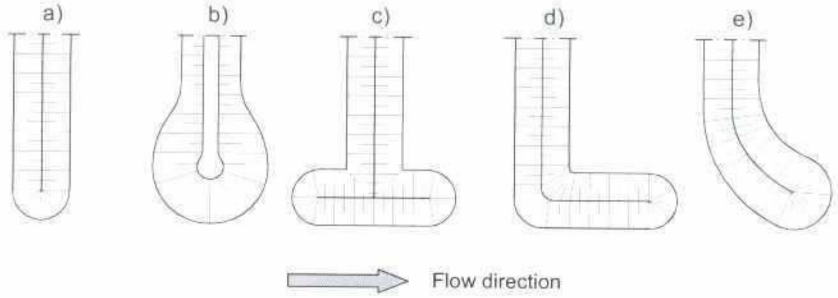


Fig. 2.2-6: Typical groyne heads

Impermeable groynes (see Fig. 2.2-7), can be built of local soil, stones, gravel and rock with suitable slopes at the shanks and the head or even vertical walls at the shanks, using steel sheet piles or pre-stressed reinforced concrete sheet piles. In case of an appropriately sloped earthdam, the trunk and the head have to be protected by a cover layer placed on a a suitable filter-layer. The main hydraulic disadvantage is the effect of flow separation at the groyne head, caused by the blockage of the flow. Therefore, special attention must be given to the toe protection at the head of the groynes, where extreme scouring occurs. In addition counter measures against the return currents, possibly attacking the bank downstream of a groyne must be considered. Falling or launching aprons have to be provided in these areas.

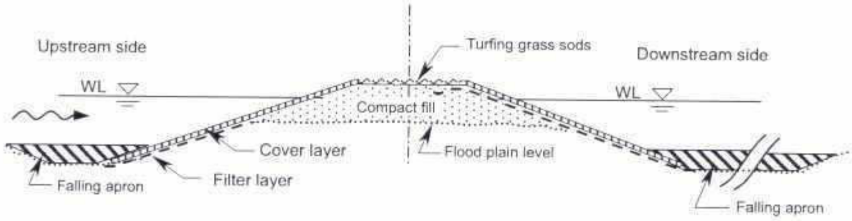


Fig. 2.2-7: Typical cross-section of an impermeable groyne (non-submerged)

For that reason, within the context of these Guidelines, impermeable groynes are not recommended as a standardized protection structure. Instead permeable groynes (see Fig. 2.2-8) are proposed, which decrease the near bank flow velocities, creating rapid deposition in that area, in particular in alluvial rivers with considerable bed load and high sediment concentration. To prevent from flow separation and to achieve a gradual deceleration of the flow velocities towards the river bank the maximum permeability chosen should be about 80% at the groyne head decreasing to 40% at the groyne root. Permeable groynes may be built of steel piles or reinforced concrete piles, which are driven into the riverbed and the flood plain, consisting of a single pile row or of several rows. In addition to current and wave attack, horizontal loads caused by floating debris must be considered in the design (length, diameter and embedment length).

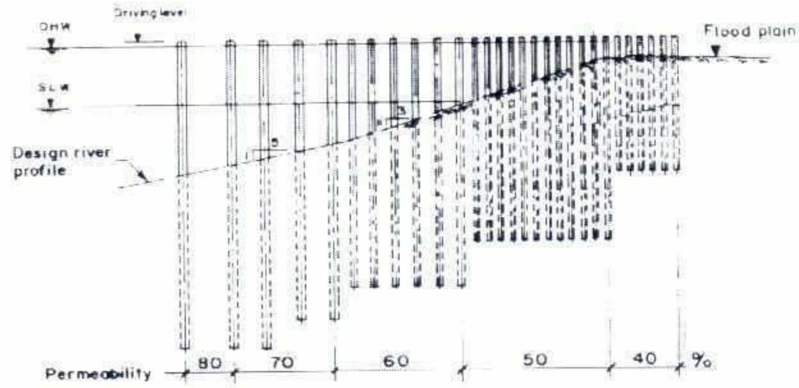


Fig. 2.2-8: Typical permeable groyne structure

The advantages of permeable groynes are evident when applied in so-called groyne fields. As mentioned before, single groynes protect the bank against erosion only over a restricted area in the vicinity of the structure and need to be rather long to be effective. Moreover, due to their sensitivity to changing directions of flow attack, single groynes are not recommended for general application. Instead, groynes should preferably be used in series, if a certain reach of the river bank is to be protected. Permeable groyne fields (Fig. 2.2-9) have the advantage of reasonably controlled and bank parallel flow pattern within the groyne field. Properly selected spacing of the groynes will enhance the effect of several permeable groynes on the reduction of the flow velocity near the bank. It has to be stressed, that in case permeable groynes are utilized, other characteristics, i.e. groyne orientation or head configuration are of minor influence regarding the hydraulic performance and stability of the structures.

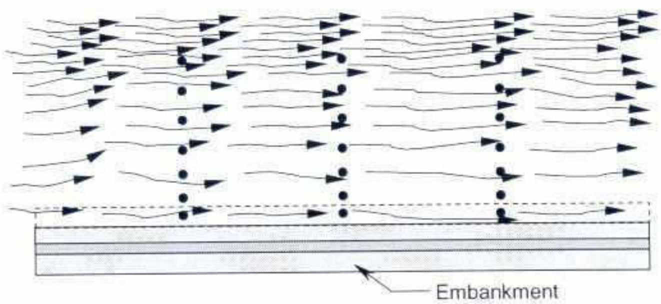


Fig. 2.2-9: Flow pattern in a series of permeable groynes

Impermeable groynes can be characterized as passive structures, followed by extreme scour at the groyne head and severe erosion of the river bank by existing return currents (Fig. 2.2-10).

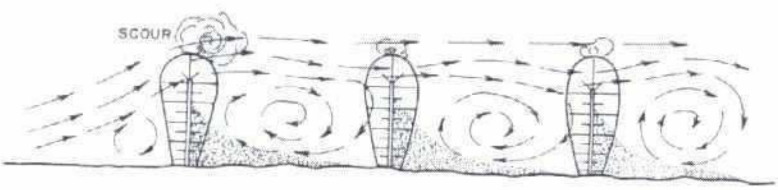


Fig. 2.2-10: Flow pattern in a series of impermeable groynes

In a standard layout of a series of groynes all the groynes should be similar regarding length, orientation and shape of the head of the groyne. However, often the most upstream groyne is attacked by the flow stronger as compared to the groynes in the main section, and in such case the design of the most upstream groyne should be adjusted by reducing the groyne length, changing the orientation or increasing the permeability.

Groynes may be also designed as submerged or gradually submerging groynes. Impermeable groynes, however, are designed non-submerged, since they are susceptible to severe erosion along the trunk by overtopping flow. Permeable submerged groynes create less severe flow disturbances as compared to impermeable groynes, nevertheless additional scour must be considered in the required embedment length. The complete or partial submergence of the structure allows for reduction of excessive pile loads induced by floating debris at higher flood levels.

2.2.4 Revetment Structures

Passive bank protection measures are primarily armoured structures or armour layers preventing a bank line from erosion but which do not create significant interference with the passing fluid. The hydraulic influence on the local flow condition is limited to changes in bed roughness. Typical passive measures are revetment structures, which are built more or less parallel to the flow to form an artificial sloped or vertical river bank. Vertical structures, e.g. retaining walls of stone-filled gabions, sheet-pile walls or similar, are not recommended for extremely mobile rivers, since these allow for high flow velocities directly at the structure toe (influence of water depth) followed by excessive scour, thus requiring substantial and expensive toe protection.

Structures, constructed on a slope of the riverbank or an embankment must be designed with a suitably chosen gradient and an adequately sized toe protection to support the revetment and to protect against scouring. Dependent on the prevalent soil to be protected against erosion and the external hydraulic loads by current and waves, many different types of revetments regarding the cross-section (berms, varying steepness, etc.) and the applied components are existent. Due to the differences between the river water level and the groundwater table additional pore pressure and subsoil is initiated, which may induce mass failure of bank lines, especially after rapid sinking of the river water level (compare Section 2.1.3).

A typical method for constructing a revetment slope is starting from a safe level above design water stage and proceeding down to the design river bed level (see Fig. 2.2-11). Considering the extreme flow conditions at the major rivers of Bangladesh, such method, involving construction of underwater slopes is not recommended in view of the required heavy floating equipment, material demands and the high costs.

Within the FAP 21 pilot an alternative was adopted and tested which is constructed completely on the flood plain area (just above SLW) and that can be build by local construction capacity. Fig. 2.2-12 shows the characteristic basic components of the recommended standardized revetment cross-section, i.e. an upper revetment, a berm, as well as launching and falling apron. The conceptual idea behind is, that the toe protection material of the completed structure - after experiencing erosion/ scouring at the structure front – will proceed ("articulate") down the developing scour slope, thus providing a continuous protective layer, which is stabilizing the riverbed/ scour slope, similar to that of an underwater construction. Due to the fact, that the vertical distance between construction level and potential scour depth in this method is rather large, substantial scour protection material must be provided.

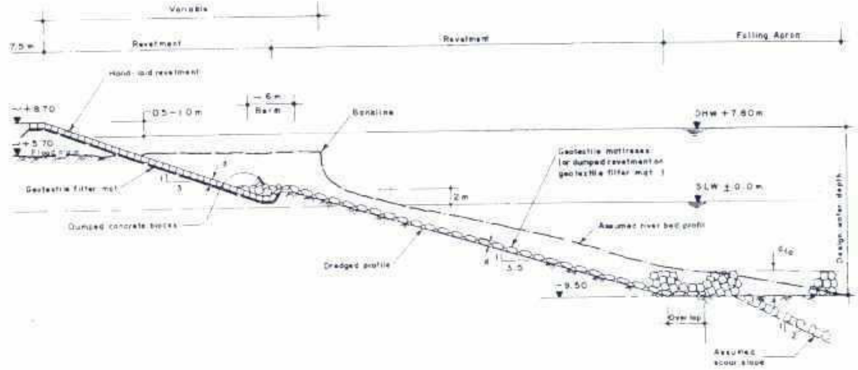


Fig. 2.2-11: Typical section of a revetment structure for under-water construction

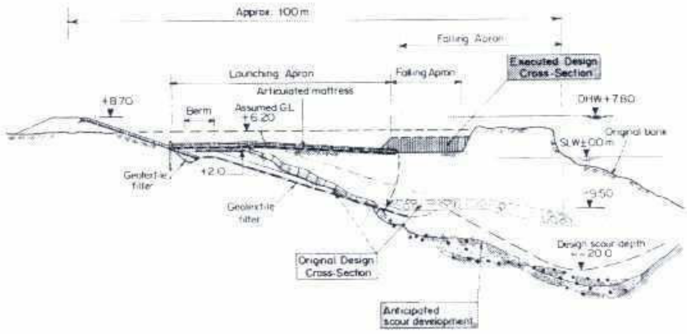


Fig. 2.2-12: Recommended revetment structure for standardized measures

84

Revetment structures must be supplemented by suitably shaped upstream and downstream terminations, as well as by a connecting dam between the revetment structure and a high level point in the near hinterland to prevent from flow- induced erosion at the inner (landward) slope of the structure.

2.3 STANDARDIZED STRUCTURES

2.3.1 Preliminary Remarks

During the FAP 21 project groyne structures and revetments were tested for several years (1995 – 2001), investigating the properties and performance of various structure components, materials and of the general layout. One fundamental issue was the introduction of **S**tandardized **P**rotection **S**tructures (SPS), where the optimal solutions based on the project findings have been considered. The advantage of standardized structures is the accelerated implementation process within well-defined budgets. Furthermore, the variety of construction materials has been restricted to certain proven products, e.g. to geo-textile filters, cc-blocks, bricks, boulders and heavy wire-mesh gabions for revetment cover layers, and tubular steel piles (in factory standard lengths) for permeable groyne fields. Therefore, material availability and construction capacity can be improved, considering suitable material and equipment stockyards at strategic locations throughout the country.

For SPS limited damages, such as surface deformations or misplacement of individual elements (individual piles, falling aprons) resulting from hydrodynamic impacts of the river, may be tolerated. Since budgets are limited, such economic approach is strongly recommended for future measures, also because a larger number protection measures can be implemented as compared to excessively over designed structures. However, the overall stability and the function of the respective structure must be ensured. It has to be emphasized that the application of SPS must be limited to certain hydraulic and morphological boundary conditions as well as to areas of certain value, but excluding locations of extraordinary importance (e.g., national projects, like Jamuna Bridge, Dhaka Town Protection). Further aspects of SPS are discussed in Chapter 3, whereas design details are given in Chapter 5 (revetments) and Chapter 6 (groyne structures).

2.3.2 Structure Classification

The standardization of future erosion protection measures requires a structure classification with respect to the expected hydraulic loads and areas of application, allowing for a simplified assessment of the risk status. To define the application of SPS and to allow for economic design four **S**tructure **C**ategories (SC) are suggested as given in Table 2.3-1. In addition the following definitions should be considered:

SC1 Minor measures and structures that can be coped by traditional means and/or ad-hoc measures.

SC2/ SC3 Erosion prevention within identified priority areas of valuable assets. Limited structural damages keeping the primary function may be tolerable, and adaptations to meet changing requirements are generally feasible.

SC4 Measures for objects of extra-ordinary importance and/or most severe and complex hydraulic and morphological conditions. Damages are not acceptable.

Structure Category	Expected Impact	Depth averaged flow velocity ū [m/s]	Design wave height H_s [m]	Total scour / water depth [m]
1	Light	< 1	< 0.25	< 10 m
2	Moderate	> 1 - 2.0	0.25 - 0.50	10 - 20 m
3	High	> 2.0 – 3.0	0.5 - 1.0	20 - 30 m
4	Very high	> 3.0	> 1.0	> 30 m

Table 2.3-1: Recommended structure categories (SC1 – SC4)

Within the proposed limitations of application Structure Category 1 corresponds to light and moderate flow and wave attack and applies for smaller rivers or branches with minor discharge. For such locations, erosion can be controlled by low cost measures.

More exposed locations are associated to the Structure Categories 2 and 3, which are the proposed categories for standardized structures (SPS) for erosion protection measures at the Jamuna and other major rivers in Bangladesh. The limits for application for SC2 and SC3 as defined in Table 2.3-1 are on a tentative basis, including a range of morphodynamic and hydraulic conditions. The range of limiting parameters should be confirmed or improved respectively by further experience along the rivers in the future.

Structure Category 4 covers sites and objects of extraordinary importance and/or exceptional and most complex hydraulic conditions. These situations call for specially designed protection structures on the basis of detailed studies, modelling, and design efforts with dedicated design and planning parameters. However, also for this structure category the design principles of the proposed standardized structures may be applied with suitable adaptation.

2.3.3 Multi-Criteria Analysis of Revetments versus Groyne Fields

Groyne structures and revetment structures can be compared with respect to significant technical and socio-economical criteria. Other criteria, like financial or environmental aspects depend largely on the local situation of the envisaged structure. Therefore, these are not considered in this Section.

The comparison, as compiled in Table 2.3-2, is restricted to standardized measures (SPS) of structure categories SC2 and SC3 and is based on the evaluation of the implemented test structures within the FAP 21 pilot project. The evaluation of the most relevant criteria have been rated qualitatively and the results are of general validity, therefore these can be transferred to other erosion protection structures of similar type. For a quantitative evaluation a weighing-factor must be introduced for each criteria, which, however, depends on the local conditions and can therefore not be assessed generally.

S. No.	Criteria	Groyne Structure		Revetment Structure	Remarks
		Permeable	Impermeable		
1. Hydraulic Loads					
1.1	Tolerant to unexpected hydraulic loads	Yes	Moderate	Moderate	
1.2	Tolerant to unexpected scouring	Yes	Limited	Limited	
2. Production with Local Materials and Skills					
2.1	Suitable for Bangladesh contractors	Yes		Yes	
2.2	Use of local materials	Possible	Largely	Largely	
2.3	Materials to be imported or reproduced in Bangladesh	Large diameter steel piles	Geo-textile filter	Geo-textile filters and mattresses	
2.4	Suitable for strategic material stock-piling	Yes	Limited	Limited	
2.5	Specialised skills	Large diameter pile installation		Mattress installation	
3. Construction Constraints					
3.1	Construction on the flood plain	Yes		Yes	
3.2	Construction into the river under moderate flow conditions	Yes	No	No	
3.3	Prone to early rise of river water level (risk of destruction of unfinished works)	No	Yes	Yes	
3.4	Site mobilisation time (the period required after award to physically start the construction)	Short [1] (Moderate)		Short	3)
3.5	Construction time	Moderate		Long	
4. Adaptability, Maintenance and Repair					
4.1	Adaptability to specific local conditions (e.g. to suit river response)	Good		Good	
4.2	Requirements for Maintenance and Repair	Low	Moderate [2] High	Moderate [2] High	
5. Socio-Economic Aspects					
5.1	Job opportunities and participation of local population during construction	Low	Moderate	High	
5.2	Land acquisition demand	Low	Moderate	High	Dependent on configuration

Notes:
1) short, if the piling equipment is available and operational, otherwise moderate
2) moderate, if the toe protection is accessible during the dry season, otherwise high
3) provided, material can be taken from strategic stock

Table 2.3-2: Multi-criteria analysis (MCA) of standardized erosion protection structures (for structure categories SC 2 and SC3)

Within the MCA a **groyne structure** is defined as a field of permeable or semi-permeable groynes, which are connected to a main embankment and extend into the river. A **revetment structure** is a reinforced embankment of a certain length with protected slopes as well as launching and falling aprons for toe protection. The impermeable part of a groyne corresponds in some design aspects more to a revetment structure than to a pile structure. Therefore, difference has been made between permeable and the impermeable groynes (or groyne sections).

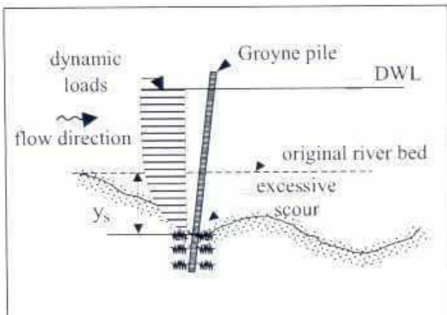
Principally, both types of structures can be also employed at locations with extremely high hydraulic loads (SC4). However, a revetment structure (and similarly the impermeable part of a groyne) must be fully completed before the water level starts to rise, as otherwise the incomplete structure is prone to fail due to undermining by river bed erosion. In contrast to this, a permeable groyne of piles can even be completed when the water level has already started to rise and it will not fail completely, even if a single pile is lost.

2.3.4 Typical Failure Modes of Revetments and Groyne Structures

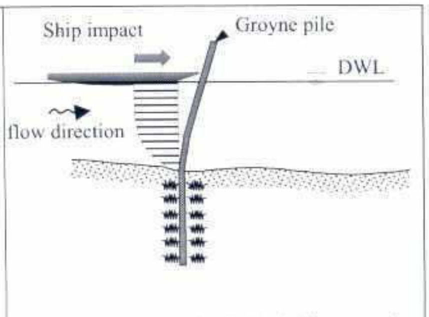
Damages at structures are initiated by different causes, e.g.

- uncertainties in subsoil conditions
- irregularities or deficiencies in material qualities
- substantial change in boundary conditions
- underestimation of design loads
- poor construction
- unintended use of structure (e.g. excessive live load)

which must be prevented as far as possible by adequate design, experienced contractors combined with appropriate construction supervision and also through peoples awareness campaigns. Nevertheless, all structure components have a certain risk potential throughout their life-time, which also depends on the quality of the monitoring activities. Some of the most typical failure are exemplary shown in Table 2.3-3 (for groyne structures) and Table 2.3-4 (for revetments). Other failures may also take place, and might have even higher priority dependent on the local situation.



Rotation of individual piles caused by insufficient pile embedment length, e.g. due to excessive scour)



Plastic deformation of individual piles caused by unexpected horizontal loads (floating debris, extreme flow velocity, ship collisions, etc.)

Table 2.3-3: Relevant failure modes of groyne structures at river banks

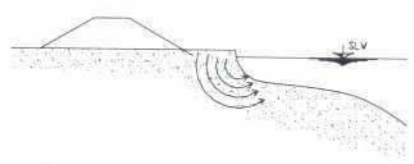
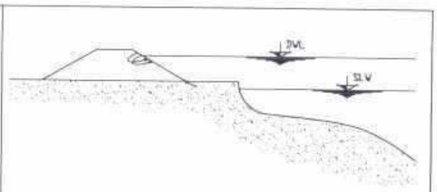
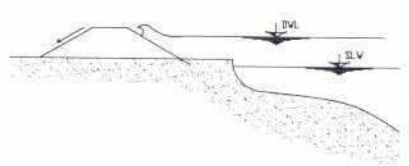
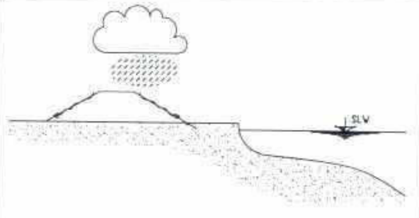
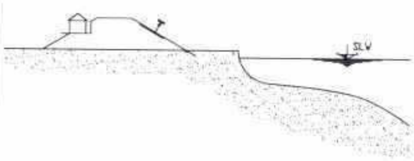
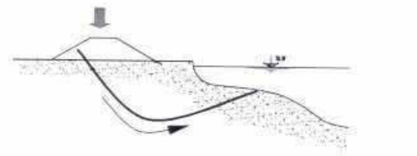
 Current induced erosion at structure toe	 Current induced erosion of the revetment cover layer
 Damages of the protective layer by wave impacts	 Erosion of subsoil and filter material by rain fall
 Removal of material and damage to the structure integrity	 Slip failure induced by excessive vertical loads

Table 2.3-4: Relevant failure modes of revetments at river banks

2.4 MULTI-CRITERIA ANALYSIS OF CONSTRUCTION MATERIALS AND METHODS

The timely and successful implementation of SPS (SC2 and SC3) requires a standardization of construction materials and methods, which should be available or producible in the country, and suit local construction capacities and capability.

Generally, the use of standardized local materials and construction methods by Bangladesh contractors are possible for groyne and revetment structures. Fabrication capacities for the production of geotextile fabrics and large steel piles can be built up in the country. Strategic fabrication and stockpiling of groyne piles could be established, but strategic stockpiling of large quantities of material needed for the construction of a revetment structure should be limited to key materials.

2.4.1 Permeable Groyne Sections

Significant criteria of steel piles, in-situ cast reinforced concrete piles and pre-cast pre-stressed concrete piles are compared in Table 2.4-1. Other common materials, like wood or plastic, are not suitable for the structure categories SC2 and SC3 and are, therefore, not considered in the comparison.

Concrete piles are restricted to moderate water depths and should consequently be applied for the landward part of the permeable groynes only. Steel piles can be designed to suit any situation with exceptional scouring depths, as they are less vulnerable to excessive horizontal loading and scouring, provided, that the embedment lengths of the piles are safely chosen.

S. No.	Criteria	Steel Piles	Concrete Piles	
			In-situ cast	Pre-cast Pre-stressed
1. Hydraulic Loads				
1.1	Vulnerable to unexpected hydraulic loads	No	No	No
1.2	Vulnerable to unexpected scouring	No	Yes	Yes
2. Production with Local Materials and Skill				
2.1	Suitable for Bangladesh contractors	Yes	Yes	Yes
2.2	Availability of material in Bangladesh	Reproducible	Available	Available
2.3	Suitable for strategic material stock-piling	Yes	Yes (basic materials)	Yes
2.4	Specialised skills	Pile fabrication, welding and large dia. pile installation	No	Pile fabrication.
3. Construction Constraints				
3.1	Installation on the flood plain	Limited by possible total driving depth	Yes	Limited by possible total driving depth
3.2	Installation into the river under moderate flow conditions	Yes	No	Yes
3.3	Site mobilisation time (the period required after award to physically start the construction)	Depends on availability of piling equipment	Short	Depends on availability of piling equipment
3.4	Installation time for single pile	Short	Moderate	Short
4. Adaptability, Maintenance and Repair				
4.1	Adaptability to specific local conditions (e.g. for repair or to suit river response)	Easy	Not possible, piles must be replaced	Very complicated
5. Socio-Economic Aspects				
5.1	Job opportunities and participation of local population during construction	Low	Moderate	Low

Table 2.4-1 : Multi-criteria analysis of groyne piles applicable for standard erosion protection measures (for structure categories SC2 and SC3)

Considering the quick implementation between two monsoon seasons, the construction window and other construction constraints are additional and very important criteria for the selection of the preferred pile system. In this context, pre-cast piles have the advantage, that they can be installed in the water even under moderate flow conditions, whereas in-situ concrete piles can only be built in the dry (on the flood plain), unless very specific floating equipment would be mobilised from abroad. Installation time for a single pile is much shorter for pre-cast piles than for in-situ piles. On the other hand, in-situ bored piles can be installed with conventional equipment, which is sufficiently available, so that several piles can be built at once, thus reducing the overall construction time.

2.4.2 Impermeable Groyne Sections

For the impermeable part of the groynes heavily protected earthdams, sheet pile cofferdams or concrete spurs may be utilized alternatively. Concrete block gravity structures are not feasible under the prevailing subsoil-conditions. Vertical structures constructed of sheet piles require a considerable embedment length, which in general does not allow for economic measures, while earth dams are more vulnerable to high flow velocities due to possible failure of the toe and slope protection. Significant criteria on the applicability of different structural measures are compiled in Table 2.4-2 and discussed in the following.

S. No.	Criteria	Earth Dams	Cofferdams		Concrete Spurs
			Steel sheet piles	CC sheet piles	
1. Hydraulic Loads					
1.1	Creation of smooth transition between im-permeable and permeable section of the groyne	Good	Limited	Limited	Not applica-ble
1.2	Favourable to unexpected hydraulic loads	Limited	Yes	Yes	Yes
1.3	Favourable to unexpected scouring	No	Limited [1]	Limited [1]	Limited [1]
2. Production with Local Materials and Skill					
2.1	Availability of material in Bangladesh	Available	Available	Available	Available
2.2	Suitable for Bangladesh contractors	Yes	Yes	Yes	Yes
2.3	Suitable for strategic material stock-piling	Yes [2]	Yes [2]	Yes [2]	Limited
2.4	Specialised skills	No	Sheet pile installation		No
3. Construction Constraints					
3.1	Construction on the flood plain	Yes	Yes	Yes	Yes
3.2	Construction into the river under moderate flow conditions	No	Limited	Limited	No
3.3	Site mobilization time	Short	Short	Short	Short
3.4	Construction time	Moderate	Moderate	Moderate	Moderate
4. Adaptability, Maintenance and Repair					
4.1	Adaptability to specific local conditions (e.g. for repair or to suit river response)	Easy	Complicated	Complicated	Complicated
5. Socio-Economic Aspects					
5.1	Job opportunities and participation of local population during construction	High	Good	Good	Good

Notes:
1) depending on the embedded length of the sheet piles / concrete structure
2) it is assumed, that the relatively small quantities of earth required for the construction are taken from borrow pits nearby and must therefore not be stockpiled

Table 2.4-2 : Multi-criteria analysis of impermeable groyne sections for standard erosion protection structures (structure categories SC2 and SC3)

Generally, impermeable groyne sections should be considered for structure category SC4 only and must undergo special design considerations. Impermeable groynes can be employed also under higher hydraulic loads, provided that they are protected against scouring by a suitably dimensioned bed and toe protection. Construction on the flood plain is preferred for any type of impermeable groyne, however, cofferdams can be built also in shallow water under moderate flow conditions. With regard to combined groynes, earth dams allow for a more gentle transition between impermeable and permeable sections of a groyne, limiting unfavourable flow conditions and the scour depth in this critical area.

Except steel sheet piles, all essential materials and production plants for the considered construction elements are available in Bangladesh. The concrete spurs are cast in-situ and, thus, only basic materials like cement, aggregates or reinforcement steel and formwork can be stockpiled.

Participation of local population in construction is best for earthdams, for carrying out most earthworks and casting and placing of concrete blocks for slope and toe protection by unskilled labour.

2.4.3 Revetments and Bed Protections

The general properties and performance of the tested materials for bank protection works are compiled in Table 2.4-3. Except some specific geo-textile fabrics (articulating and collapsing mattresses), all materials are available in Bangladesh or could be produced within the country. Further aspects focussing at the applicability of the protective systems with regard to hydraulic loads and other criteria are summarised in Table 2.4-4. Some additional information

Revetments must provide a sufficient permeability to avoid uplift forces from groundwater pressure. A high permeability of the revetment is also favourable from the environmental point of view, as it avoids a separation of the terrestrial and aquatic biospheres. For that reason cover layers with a continuous geo-textile fabric were ranked with a low environmental suitability, as they separate the subsoil from the aquatic zone. In addition a high flexibility of protection systems (filter and cover material) is of advantage with respect to potential settlements of the soil and subsequent deformation of the structure as well as to impacts during construction. Hand pitched cc-blocks are the recommended protection system for the slopes of standard revetment structures above the water line. Concrete blocks are more durable than bricks and less attractive to pilferage. If well dimensioned, they are able to suit any flow condition. Sizes of 30-50 cm have proven to be sufficient to withstand the hydraulic loads occurring at the Jamuna river.

S.-No.	Structure Type	Origin	Flexi-bility	Permea-bility	Mainte-nance Require-ments	Environmental Impact	
						Visual	Ecological
1. Rock / Boulders							
1.1	Rip-rap	Bangladesh	High	High	Low to Medium	Acceptable to Good	Acceptable to Good
1.2	Hand pitched stone	Bangladesh	Low	Low	Medium	Acceptable to Good	Poor to Acceptable
1.3	Cement grouted stone	Bangladesh	Low	Low	Low to Medium	Poor to Acceptable	Poor
1.4	Bitumen grouted stone	Bangladesh	Medium	Low to Medium	Low	Poor to Acceptable	Poor to Acceptable
2. Gabions							
2.1	Mattresses (brick or stone fill)	Reproducible in Bangladesh	High	High	Low to Medium	Acceptable	Acceptable
2.2	Box Gabions (stone/rock fill)	Reproducible in Bangladesh	Medium	High	Low to Medium	Poor to Acceptable	Acceptable
2.3	Gabion Sacks (stone/rock fill)	Reproducible in Bangladesh	Medium to High	High	Low to Medium	Acceptable	Acceptable
3. Concrete (pre-cast units)							
3.1	CC-slabs	Bangladesh (site production)	Medium	Low	Low	Poor	Poor
3.2	CC Interlocking slabs	Bangladesh (factory make)	Medium	Low	Low	Poor	Poor
3.3	Hand pitched CC-blocks	Bangladesh (site production)	Medium	Low	Low	Poor	Poor
3.4	Dumped CC-blocks	Bangladesh (site production)	High	Medium to High	Low	Poor to Acceptable	Poor to Acceptable
4. Articulating Mattresses							
4.1	Gabion mattresses, steel wire linked (with stone fill)	Reproducible in Bangladesh	High	High	Medium	Acceptable	Acceptable
4.2	CC-blocks attached to geotextile filter mat, steel wire linked	In-situ cast, geotextile to be imported	High	Medium	Medium	Acceptable	Poor
4.3	Tubular geotextile fabric mattress; sand or bitumen-sand filled	To be imported	High	Medium	Medium	Poor	Poor to Acceptable
4.4	Collapsible geotextile block mattress, sand filled	To be imported	High	Medium	Medium	Poor	Poor to Acceptable
4.5	Collapsible Concrete Block Mattress	To be imported	Low	Low	Low to Medium	Poor	Poor
5. Sand Containers							
5.1	Geotextile-Sand bags (up to 250 kg)	Reproducible in Bangladesh	Medium	Low to Medium	Low to Medium	Poor	Poor
5.2	Geotextile-Sand con-tainers (up to 900 kg)	Reproducible in Bangladesh	Medium	Low to Medium	Low to Medium	Poor	Poor
6. Bio-Engineering							
6.1	Durba grass sods on Geo-Jute Soil Saver	Bangladesh	High	High	High	Good	Good
6.2	Vetiver plantation	Bangladesh	High	High	Medium	Good	Good

Table 2.4-3 : Multi-criteria analysis of materials for protective layers for standard erosion protection measures (Loading Classes 2 and 3)

Articulating mattresses are suitable as launching aprons for the toe protection of revetments. They are preferably installed on the flood plain, but can also be installed in shallow water, using purpose made installation frames. The mattresses must be anchored at the toe of the upper revetment. Concrete block and gabion mattresses are a very effective protection systems. Box gabions consist of wire mesh boxes filled in-situ with bricks or natural stones, but can also be pre-filled and transported to the site, which, however, requires heavy and specialized equipment. Locally produced wire mesh gabions showed significant signs of corrosion and insufficient strength and integrity. It is recommended to use only wire mesh of high quality, preferably machine made with PVC-coating. In particular, when applied as launching aprons the mattress systems must be highly durable and flexible to follow potential erosion of the riverbed near the bankline and stabilise the developing under water slope. Gabion-mattresses and concrete block mattresses can be produced to a large extent from local materials.

Well dimensioned rip-rap layers can be utilized for slope and bed protections even under high flow attack. Installation under water is generally possible but requires special methods to suit conditions similar to that of the Jamuna. For short bank reaches and for the repair of existing revetments, hand pitched stones are suitable. In case of extreme hydraulic loads and slopes steeper than 1V : 2H, grouting is recommended to increase the stability.

Geotextile sand containers, which can be tailored locally, are simple to be installed and can be used for toe protection. Due to their suitability for strategic stockpiling, they are ideal for ad-hoc repair measures applicable for above and below water level installation.

For slopes with minor flow attack, natural protection by Durba grass or Vetiver may be the favourable solution. Durba grass is preferably used for the land-sided slope of main embankments or for the upper area of river-sided slopes. Due to stronger rooting, Vetiver is a suitable material for toe protections under low-flow conditions.

The requirements for maintenance and repair are in an acceptable range for all investigated systems. Generally, all systems need regular inspections and fast action in case of observed damages, to avoid progressive destruction. Slope protections by Durba grass sods (mainly used at the inner slopes) are particularly vulnerable to damages by human intervention or animals and need therefore more intensive observation and nursing during the first years. However, repair is simple and can be done with local labour at low cost.

S. No.	Structure Type	Loading Class				Bank Slope			Main Applicability
		1	2	3	4	<1:2 V:H	> 1:2 V:H	Near Vertical	
		+ = Recommended				- = Not recommended			
1. Rip-rap									
1.1	Rip-rap	+	+	+	-	+	-	-	• bank and bed protection • installation above and below water level
1.2	Hand pitched stone	+	+	+	-	+	+	-	• short bank reaches • repair of existing re-vetments • installation above water level
1.3	Cement grouted stone	+	+	+	+	+	+	-	• areas of attack by strong currents • installation above water level
1.4	Bitumen grouted stone	+	+	+	+	+	+	-	• areas of attack by strong currents • installation above water level
2. Gabions									
2.1	Mattresses (brick or stone fill)	+	+	+	+	+	-	-	• bank protection of large areas • installation above water level
2.2	Box Gabions (stone/rock fill)	+	+	+	+	+	+	+	• retaining wall for bank protection • installation above water level
2.3	Gabion Sacks (stone/rock fill)	+	+	+	+	+	-	-	• toe protection • installation above and below water level
3. Concrete (pre-cast units)									
3.1	CC-slabs	+	+	-	-	+	+	-	• slope protection • installation above water level
3.2	CC-Interlocking slabs	+	+	-	-	+	+	-	• slope protection • installation above water level
3.3	Hand pitched CC-blocks	+	+	+	+	+	+	+	• slope protection • installation above water level
3.4	Dumped CC-blocks	+	+	+	+	+	+	-	• bed and bank protection in case of strong current and wave attack • installation above and below water level

Table 2.4-4 : Applicability of materials for revetments and bed protections
(table continued next page)

S. No.	Structure Type	Loading Class				Bank Slope			Main Applicability
		1	2	3	4	<1:2 V:H	> 1:2 V:H	Near Vertical	
		+ = Recommended				- = Not recommended			
4. Articulating Mattresses									
4.1	Gabion mattresses, steel wire linked (with stone fill)	+	+	+	-	+	+	-	• launching apron • slope protection • construction above water level (limited water depths) (1)
4.2	CC-blocks attached to geotextile filter mat, steel wire linked	+	+	+	-	+	+	-	• launching apron • slope protection • construction above water level (cast in place) (1)
4.3	Tubular geotextile fabric mattress; sand filled or bitumen-sand filled	+	+	-	-	+	+	-	• launching apron • slope protection • installation above water level (1)
4.4	Collapsible sand filled geotextile mattress	+	+	-	-	+	+	-	• launching apron • slope protection • installation above water level (1)
4.5	Collapsible concrete filled geotextile mattress	+	+	+	-	+	+	-	• launching apron • slope protection • installation above water level (1)
5 Sand Containers									
5.1	Geotextile-Sand bags (up to 250 kg)	+	-	-	-	+	+	+	• falling apron and toe protection • installation above and below water level
5.2	Geotextile-Sand containers (up to 900 kg)	+	+	+	-	+	+	+	• falling apron and toe protection • installation above and below water level
6. Bio-Engineering									
6.1	Durba grass sods	+	-	-	-	+	-	-	• upper reaches of banks above mean water level • preferably on land-side • on river-side prone to wave erosion • installation above water level
6.2	Vetiver plantation	+	+	-	-	+	-	-	• toe protection to upper reaches of banks • installation above water level

Note: 1) Articulating mattress systems can generally be installed below water level, but require special floating equipment that is presently not available in Bangladesh. With the prevalent flow conditions construction of under-water slope protections should be avoided for erosion protection structures.

Table 2.4-4: (continued): Applicability of materials for revetments and bed protections

3 GENERAL PLANNING AND IMPLEMENTATION ASPECTS

3.1 APPROACH

The conventional planning procedure for the construction of protective structures as given in Fig. 3.1-1 is starting from the evaluation of the existing hydraulic and morphological situation and the expected future development (input design parameters), followed by the evaluation of potential optimal solutions (conceptual design), aspects related to the actual construction and the operation as well as the maintenance scheme of the structure.

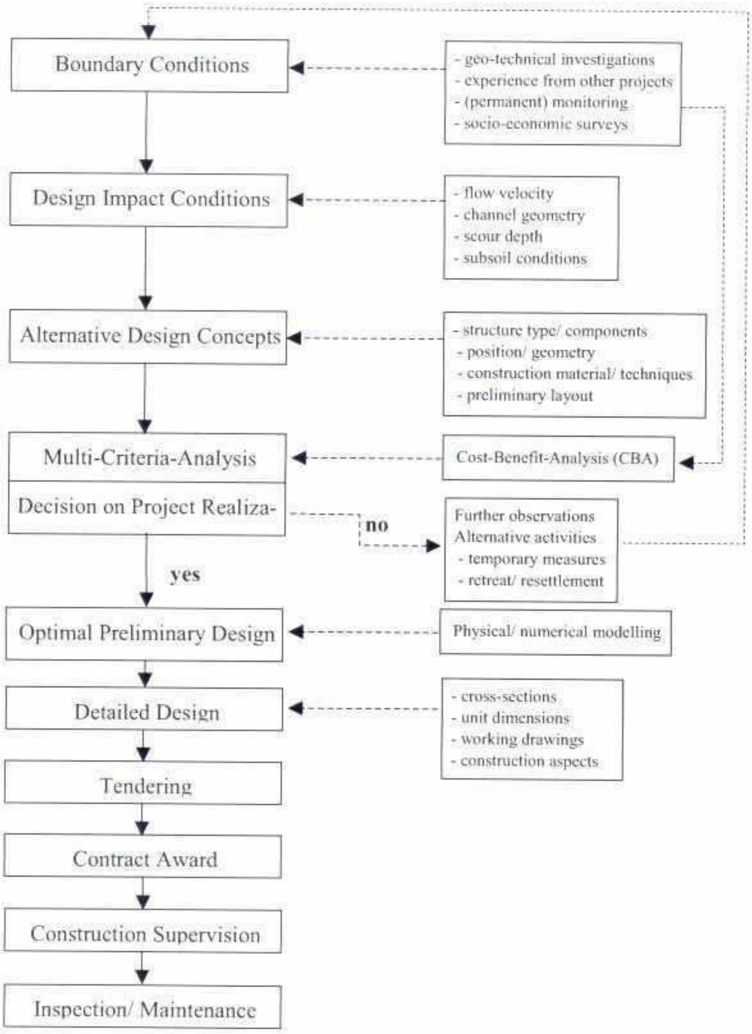


Fig. 3.1-1: Key aspects of the conventional planning track for river bank protection measures

Normally, the design of erosion protection structures require decent investigations for each specific location ("case studies"), evaluating different structural solutions ("conceptual design") and the interference between structures and impact forces ("optimisation"). For that reason, theoretical investigations as well as physical and numerical modelling is used to substantiate structural options, with regard to functional and safety aspects.

Considering the harsh environment at the major rivers in Bangladesh regarding the mobility of the river bed (which can be observed in particular at the Jamuna river) and the extreme variations in discharge as well as the fact, that a comprehensive investigation of the existing morphological and hydraulic boundary conditions is generally a very time consuming process, the successful implementation of individual projected countermeasures against bank erosion is put at risk.

In this regard, the experience during the implementation of the FAP21 pilot structures revealed two major constraints:

- the extent of preparatory studies, which was strongly related to the research character of the project, but, in view of extreme erosion rates and the restricted construction window, time consuming preparatory studies will remain a decisive problem in the planning process of future projects
- delays in the structure implementation, which were hampering the working progress, which were mainly related to land acquisition, to inadequate construction techniques and to delays regarding the supply of construction materials

These constraints led to the attempt to improve certain steps of the planning process of bank protection structures to allow for a rapid and safe planning frame work and a fast construction of the respective measures against bank erosion in future.

The key issues, which are covered by a new strategic approach, the **S**trategic **M**aster **P**lan (SMP), can be summarized as follows:

- Introduction of **S**tandardized **P**rotection **S**tructures (SPS)
- Optimization of the preparatory planning (from case studies to methodical investigations)
- **S**trategy for **I**dentification of **P**riority **P**rotection **S**ites" (SIPPS)

These aspects are discussed in the following Sections.

3.2 APPLICATION OF STANDARDIZED PROTECTION STRUCTURES

The introduction of **S**tandardized **P**rotection **S**tructures (SPS) is one of the focal issues of the SMP, because it provides the basis for an economic and safe design, when serving a predefined range of boundary conditions (structure categories SC, see Chapter 2). Furthermore SPS are advantageous in terms of:

- simplified and accelerated planning process
- use of standardized components (material quality and availability)
- fast construction (availability of equipment/ experience with construction techniques)

The utilization of standardized structures offers significant advantages when compared to individually planned measures against bank erosion, because the normally required assessment of different conceptual design solutions is restricted to relatively small modifications/adaptations to take into account specific boundary conditions at a potential site location. The accelerated planning process is very important, especially after confirmation of the proposed project by the decision makers, because the time period between (positive) decision and implementation of a project is considered to be rather short (i.e. approx. 5 months during dry season).

In addition, standardized structural elements, materials and construction equipment can be produced or procured sufficiently before construction start (delays in material supply might considerably affect the construction schedule and thus put the uncompleted structure at risk). Nevertheless, the availability of actual dimensions and quantities of material needed has therefore to be confirmed during the planning process.

The frequent application of standardized structural solutions against bank erosion will further increase the respective experience and know-how, provided that the structure behaviour and flow conditions are monitored and evaluated sufficiently after project completion.

Due to the high mobility of braiding and anabranching rivers, the conventional approach would have to include the superposition of the most unfavourable conditions, which would lead to an over-designed structure with consequent excessively high investment costs (uneconomic design, see Table 3.2-1). Experience from other structural measures shows that even a massive structure might be inadequate to accommodate unexpected extraordinary loading, or if the chosen position or the alignment is sub-optimal.

Fictitious structure life time (monsoon seasons)																														Design
1-10										11-20										21-30										
				M					M						M							M								Uneconomic
				M						M			**M**			M			**M**			M				M				Acceptable
		M			**M**			M			**M**			M			**M**			M			**M**			M			**M**	Optimal
		M						**M**	**R**	**A**			**M**				M					M				M			**M**	Adaptable

Actual loads « design loads; Actual loads = design loads; Actual loads > design loads; M: maintenance; R: repair; A: adaptation

Table 3.2-1: Time and durability relationship over a fictitious structure life (schematic)

Standard Protection Structures (SPS) are designed at a lower safety level as compared to the so-called "hard point" structures. To avoid uneconomical over-designing against maximum conceivable hydraulic loads, the expected hydraulic conditions of the forthcoming flood seasons at an identified location are used as design basis. Taking into account the difficulties in predicting the morphology of e.g. the Jamuna river, particularly over larger time periods, the assumptions regarding the actual operating time of a structure as well as the prevailing river channel and flow properties and the structure-flow interference within the structure life time are rather vague during the design process.

Therefore, an economically more viable approach is recommended, which allows for a substantial reduction of the initial investment, but may require slightly higher maintenance costs, and in some cases an adaptation of the original layout. Also an economic design approach will certainly include some monsoon periods, where smaller hydraulic loads occur as compared to the design values. On the other hand, in some periods extraordinary situations may occur, affecting the integrity of individual structure components, but not the intended function in terms of bank protection (acceptable design). If the ratio of **structure service life** (for which the actual hydraulic loads are in the range of the design loads, green shading in Table 3.2-1) versus **structure lifetime** approaches unity, the theoretical optimal design in economic terms is achieved. However, a certain probability of unexpectedly high loads (versus the initial design assumptions) within the structure service life can be tolerated. Therefore, structures that can be adapted and reinforced without difficulties after completion are to be favoured and are economically most feasible.

3.3 PLANNING STRATEGY

The socio-economic importance of a location and the expected erosion impact (defining the potential "risk" at a specific location) is affecting the applied design parameters and safety margins. Furthermore, dependent on the priority of an area as well as on the existing/ expected morphodynamic and hydraulic boundary conditions, different planning intensities are required, which can be roughly determined by the respective Structure Category, as shown in Fig. 3.3-1.

Certain locations with minor flow attack or of minor importance (SC 1) should be subjected to further observation and, if necessary, limited action should be based on traditional measures, which do not involve intensive preparatory planning efforts. The utilization of standardized structures is restricted to moderate or high flow/ erosion attack (SC 2/3). The detailed planning and design procedure of SPS as well as aspects related to inspection and maintenance and other information is given in Chapters 4 to 9 of this handbook. With regard to SPS, detailed planning and design stands for pre-worked design sheets, which allow for a very short duration from start of the design process towards the completed tender documents.

Definitely, also for SPS, a detailed design process would allow for further refinement of the proposed solutions. The use of physical and numerical models can contribute considerably to the understanding of the physical processes involved and help to improve the prediction of effects of structural measures. For acceptable quality of model results, calibration, i.e. the comparison between measured/calculated model results and field measurements for identical test conditions is required. The deviations between model and field can be used for adjusting model coefficients or model boundary conditions. It is recommended to also verify the adjusted model with field data, which have not been used for the calibration, to get confidence in the generality of the model.

Physical models are normally capable of giving qualitatively good simulations of the prototype conditions. Physical models are advantageous if hydraulic parameters should be obtained. An important aspect in the interpretation of results is formed by the inherent errors due to scale and laboratory effects. The translation from qualitative to quantitative results requires interpretation of an experienced model expert. Tests with a movable bed (morphological studies) are even more complex, because a proper scaling of all processes involved is not possible (distorted model). Nevertheless, it is a good tool which allows for systematic variation of significant boundary conditions.

Numerical models can be used for the same objectives as physical models and require the same calibration and verification efforts. The calibration has to be performed by a numerical expert because the use of "tuning knobs" has to be done in view of the respective physical processes involved. For studying the sensitivity of the different modelling components, a systematic variation of the involved coefficients is mandatory. Also numerical models are influenced by the inherent errors due to numerical effects, which has to be considered during the interpretation of the results. The interpretation of numerical results reflects the expertise of the modeller. Numerical models are advantageous, if the hydraulic and morphologic performance of different structure alternatives should be compared (differential analysis), but particularly for rivers showing a very high mobility and large sediment loads, the existing models need further calibration and refinement.

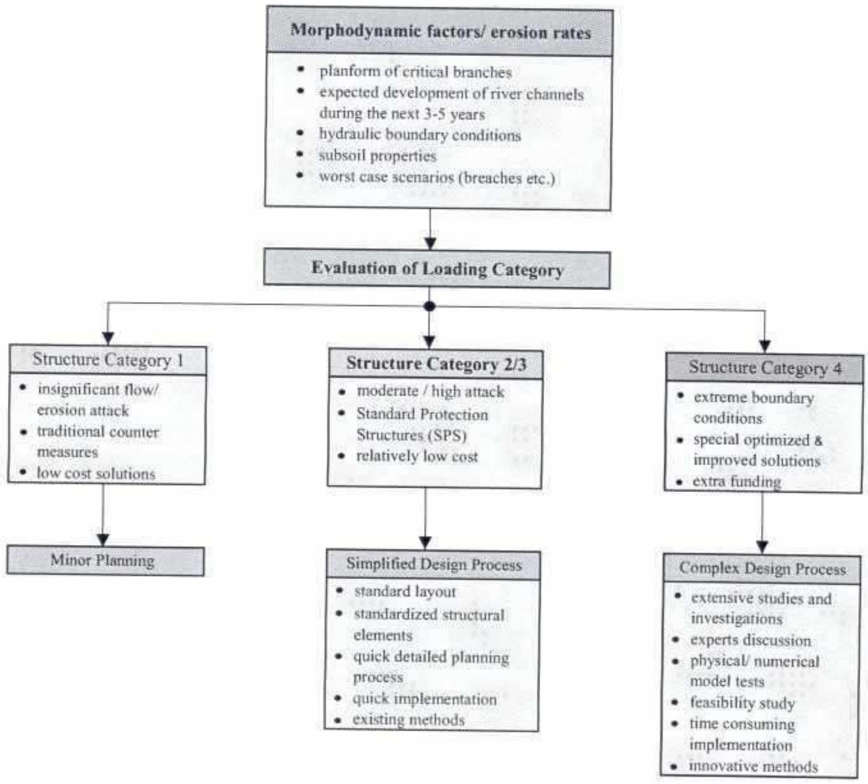


Fig. 3.3-1: Decision on planning intensity for individual bank protection measures

Due to the rather time consuming process, which might produce substantial delays in the implementation of individual countermeasures, it is strongly recommended to further develop and calibrate the existing models parallel to the implementation of future bank protection measures. Particularly for the matter of physical models it seems to be advantageous to carry out systematic tests (systematic variation of influencing parameters and boundary conditions) instead of case studies, to obtain more general results, which must be verified by data gained from the proposed comprehensive monitoring scheme (see also Chapter 7). This systematic approach, which should be carried out parallel to the implementation of various bank erosion protection measures, will contribute substantially and more effective to the further refinement of SPS, as compared to individual case studies.

Nevertheless, besides these systematic investigations, involving and combining all research methods by thorough analysis, at certain general priority locations, where extreme flow attack and/ or erosion is expected (SC 4), a comprehensive evaluation of possible morphological developments and of an optimized structural layout is required. In contrary to the simplified design process, detailed case studies involving national/ international experts (national expert panel, task force) supported by physical and numerical modelling and extended site investigations are mandatory.

In Bangladesh, the River Research Institute (RRI) has facilities and experience in physical modelling and also the Bangladesh University of Engineering and Technology (BUET) has some research facilities at smaller scale. For numerical model applications, the Surface Water Modelling Centre (SWMC) is a competent partner. The strong co-operation between experts regarding the evaluation of field data (monitoring) and for physical/ numerical modelling is obligatory to allow for reliable and accurate prediction methods.

3.4 STRATEGIC MASTER PLAN (SMP)

3.4.1 Key Objectives

A Strategic Master Plan (SMP) is mandatory to organize and co-ordinate the temporal and spatial implementation process. Within this context the term implementation includes all relevant procedures of planning, designing, constructing, monitoring and possibly of adapting a bank erosion protection structure. Suitable interfaces should ensure co-ordination and integration with the National Water Management Plan. The implementation process has to be performed in all directions taking into account new information from either direction as a basis for the decision makers at national and regional planning level. The exchange of know-how and information is a central aspect in an optimized planning process, to allow for weighing up of different locations where bank protection is required and to co-ordinate immediate actions in cases of unexpected severe attack on banks near facilities of importance.

The successful implementation of bankline protection measures requires an optimal planning of the different preparatory and succeeding working steps as well as the co-ordination between different planning levels (authorities) involved.

3.4.2 Temporal and Spatial Frame of Project Implementation

The temporal and spatial implementation process of a bank protection structure is shown schematically in Fig. 3.4-1. The planning units as well as the temporal sequence may be used as a guideline. The period for the identification of general priority locations and other items is rather hypothetically, because these are continual activities. The time frame given is a rough estimate in relation to an individual project planning. Due to the high mobility of the major rivers and the probability of occurrence of extreme hydraulic events, the process needs a continuous review and appropriate adaptation. Thus, a competent organization is a must and the core of a comprehensive and flexible arrangement of any key actions to be taken within the SMP.

Some aspects of the SMP are operating in an overlapping mode between individual parallel or subsequent activities as shown in Fig. 3.4-1. For example, strategic stockpiling of construction material is not pre-arranged only for a very specific location and bank protection project, but must be available prior to construction start or to any required emergency repair or maintenance measure. Hence, the strategic planning and maintaining of countrywide material stockpiles will be an important and permanent task within the SMP. Moreover, it is a prerequisite for an effective SMP to provide sufficient financial resources for bank protection measures, to avoid unacceptable delays within the implementation process.

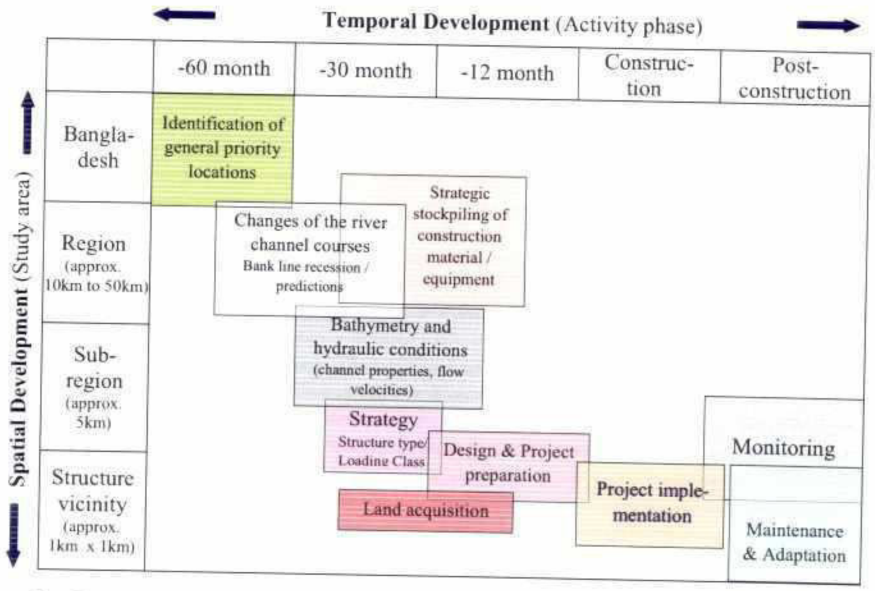


Fig. 3.4-1: Temporal and spatial development within the implementation process (SMP)

3.5 IDENTIFICATION OF PRIORITY SITES (SIPPS)

3.5.1 Data Collection and Monitoring

To allow for an efficient management and a selection of the most urgent bank protection works, a "Strategy for Identification of Priority Protection Sites" (SIPPS) is crucial. Erosion prevention is always associated with considerable capital investment. Protection measures in a specific area are rational only, if a certain accumulation of population or assets is evident or if larger scale strategies are followed. To allow for a risk assessment within the planning process, it is therefore important to introduce the socio-economic characteristics of potentially affected areas in the active flood plain of major rivers. This has got implications on the focus of the monitoring scheme, which should be directed at certain vital areas along the river stretches.

A general strategy towards safe and cost effective measures against bank erosion at the major rivers of Bangladesh requires a large-scale framework, covering the entire projected area. Regional and sub-regional planning units, responsible for monitoring restricted smaller sections of a specific river will contribute a large part of the morphological and hydraulic database.

A comprehensive assessment of existing information and anticipated developments is the key input at every level and stage of the implementation process. The data collection process and the scope and extent of the individual project stages are summarized in Fig. 3.5-1. The data collection and assessment programme includes different temporal and spatial components to provide the essential information for the SMP. The general intention is to increase the monitoring intensity with decreasing spatial scale (compare Fig. 3.5-1). The inputs from data collection and monitoring form the basis for the evaluation and comparison of different locations within the general priority classification (SIPPS).

3.5.2 Socio-economic Surveys

Priority locations in general terms are defined by a major individual asset (power plant, irrigation pump station, etc.) or by an accumulated number of assets (e.g. a village serving as a trade centre) located in the peripheries of a major river. These areas can be identified by GIS-mapping or by traditional/conventional methods combining topographical and socio-economical information, which have to be updated frequently in a priority map.
An assessment of the socio-economic impacts of proposed works is required at an early stage of the project including active participation of the concerned people. The socio-economic surveys of the river peripheries (AFP and a certain margin) have to be updated frequently, dependent on the area specific development regarding population density, economic viability, etc. Supplementary information could possibly be retrieved from data sets established by the Bangladesh Bureau of Statistics and other governmental authorities or non-government organisations (NGO). Relevant aspects for the estimation of prevented damages and associated developments as well as the impacts due to the implementation of the protective structures must be considered:

- benefits due to prevention/ reduction of erosion damages (housing, infrastructure, agriculture etc.)
- benefits due to prevention of productivity losses
- problems related to land acquisition needed for construction works
- resettlement of households, etc.

An example data sheet for a household questionnaire which could be used for socio-economic surveys can be found in Chapter 9. The results should be analysed and cross-checked, to allow for reliable information.

3.5.3 Morphological Surveys and Hydraulic Analysis

The decision on the structure category is certainly also based on the expected morphological and hydraulic impacts at a specific location, considering the respective importance. The magnitude of "risk" is dependent on the area specific socio-economic prosperity and the related probability of damage. To assess the probability of extreme bank erosion the major and most frequent monitoring activities should be focussed (but not limited) to the pre-selected priority locations.

Generally the predicted bankline development and the design boundary conditions are supported by physical and numerical modelling. Due to the complex structure and extremely high sediment transport rates of the major rivers of Bangladesh, model testing will contribute considerably to the total planning time. Therefore, - following a mid-term strategy - the pre- and post construction monitoring was promoted, to partly replace the actual modelling input. An exception in this regard must be allowed for cases, where extreme loading conditions are expected or at specific locations of vast importance (SC4).

After identification of general priority locations ("areas of accumulated assets") based on socio-economical surveys, morphological investigations covering a larger area of approx. 10 - 50 km, mostly upstream from the respective location, must follow to allow for predictions regarding the risk of bank erosion during the subsequent years. For this, a systematic record and analysis of bank line changes should be carried out on basis satellite images (EGIS).

At this stage the possible course of major branches (for anabranching or braiding rivers) and the identification of the main or decisive channel for the respective location must be determined.

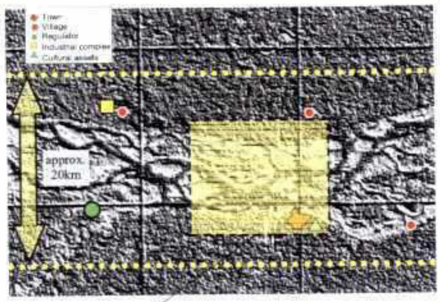
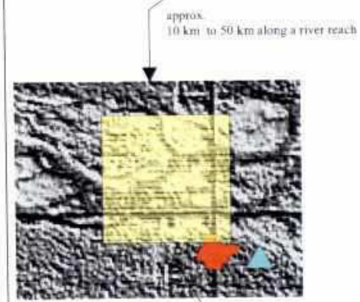
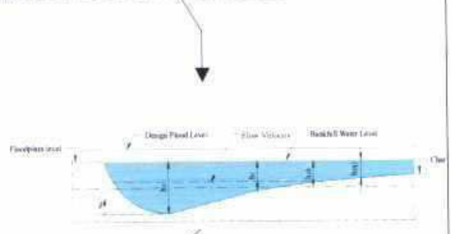
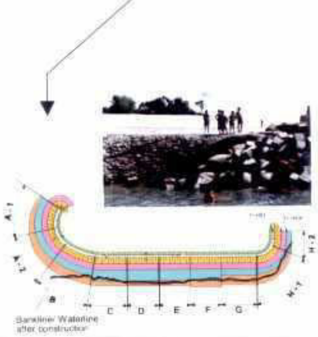
	Description	Example
Definition of general priority	**AREA:** Peripheries of the major rivers of Bangladesh (bank full river surface plus approx. 5km parallel to the river course) **SCOPE:** Definition of priority locations/ areas of high socio-economic value as general basic information for the Strategic Master Plan (SMP) **PARAMETERS:** • Population density • Industrial complexes • Trade centres • Agricultural production • Cultural assets / national heritage	
Conceptual design	**AREA:** At least 10km to 50km (mostly upstream of a potential area to be protected) **SCOPE:** Estimation of bankline and river channel development (annual reporting) in the vicinity of a general priority site which is possibly endangered in the following monsoon seasons **PARAMETERS:** • Frequent monitoring of banklines incl. major branches • Identification of main channel approach • Evaluation of erosion rates • Estimation of hydraulic conditions	
Design / Implementation	**AREA:** At least 5km (in the vicinity of a potential structure location) **SCOPE:** Estimation of hydraulic and morphological boundary conditions. Start of detailed design process. Optimal structure location and layout **PARAMETERS:** • Bathymetry • Flow velocities • Scour location and depth • Loading class / structure type • Area required for structure implementation	
Post-Construction	**AREA:** Structure components and nearfield **SCOPE:** Inspection of structural conditions as basic information for maintenance, repair or adaptation measures. Investigation of failure modes and causes to further improve the design and technical knowledge **PARAMETERS:** • Structural behaviour • Near field flow conditions	

Fig. 3.5-1: Collection of project data within future bank protection programmes

Besides using available satellite images, results from bathymetric and topographic surveys must be taken into account for the prediction of likely developments in the succeeding years. If no mathematical erosion prediction model is available, the expected changes of the bankline in the succeeding years should be estimated by graphical methods performed by an experienced river morphologist. The estimated channel developments should be covered in annual reports including also a comparison between predicted and occurred morphological changes.

The data shall be kept in a convenient format (which should be identical or generally comparable for different districts, see also Table 3.5-1) and a continuous, but at least yearly updating shall be done.

All co-ordinates should refer to the Bangladesh Transverse Mercator (BTM, horizontal datum) and the Public Works Department (PWD, vertical datum).

Village/ Town/ Other:			**Thana:**				**District:**		
Name of River:									
Sl. No	Description		Base-year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Re-marks
1	Location co-ordinates of bankline								
2	Erosion category after prediction model analysis	Cat. 1							
		Cat. 2							
		Cat. 3							
3	Co-ordinates of bankline with erosion probability exceeding	90%							
		50%							
		10%							
4	Likely affected installations	Farmland							
		Homesteads							
		Roads/ Embankments							
		Other installations							
5	Ground-thruthed bankline using topographic and bathymetric survey								

Category 1: Heavy erosion with yearly erosion rates over 100 m
Category 2: Moderate erosion with yearly erosion rates ranging between 50 and 100 m
Category 3: Low erosion with yearly erosion rates less than 50 m

Table 3.5-1: Datasheet for evaluation of erosion areas (example)

If bank erosion is expected in about the next 3-5 years, a frequent monitoring of the bankline, including bathymetric cross-sections of the decisive channels is mandatory. To get basic design data, also the prevalent flow velocities should be recorded, providing horizontal and vertical velocity profiles. If possible, velocity measurements should be done during monsoon season (bankfull discharge), otherwise extrapolation is needed, to estimate the design conditions. Furthermore, scour pattern should be assed, in particular if a channel bend is propagating towards a general priority location.

These inputs should allow for a decision on the required structure category and - in case of SC2/3 - for the most advantageous structure type (groyne field or revetment). On this basis also aspects regarding the structure length, the optimal alignment and the required land for structure implementation can be derived.

The data from post-construction monitoring and inspection are fundamental for the maintenance and repair scheme. Moreover, additional information on the structure flow interaction will contribute to a sound knowledge of the processes involved and to improved design methods in the future.

3.6 DECISION PROCESS

3.6.1 Multi-Criteria-Analysis (MCA)

In the definition stage (Phase 1, Fig. 3.6-1), structural options should be studied. Decisions on the structure type (i.e. revetment or groyne field) and the standardized structural components can be obtained under consideration of the pros and cons regarding the site-specific boundary conditions as described in Chapter 2.

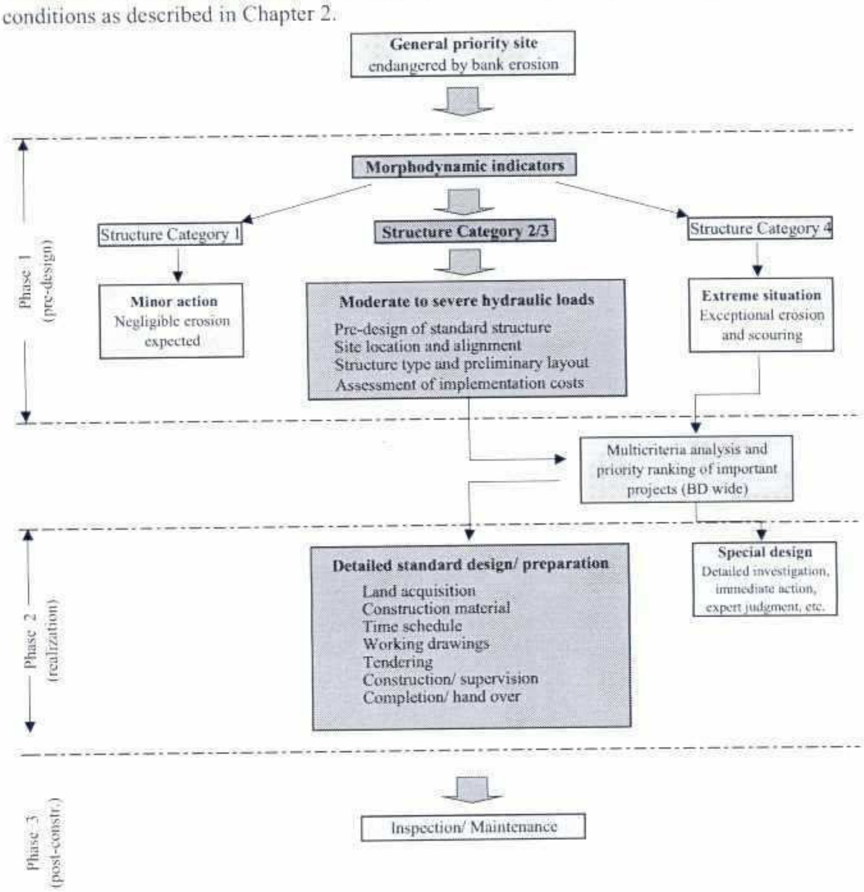


Fig. 3.6-1: Multi-criteria assessment during the planning process of individual bank protection measures

Simultaneously with the conceptual definition of a structural solution for an identified location, the channel properties and hydraulic conditions have to be further monitored to enhance the prediction of the likely development of the riverbank within the subsequent years. The proposal on the preferred structure type including the expected capital costs and assessed benefits is used as input for the MCA.

The weighing scheme of different criteria related to socio-economic, political, environmental and technical aspects should be defined by an expert group, which is also needed for the preparation of the decision base as well as consultancy to the decision makers. Generally, the Multi-Criteria Analysis is a repetitive process over time and must be carried out on the basis of continuously updated data sets to achieve a substantial reliability in the priority ranking.

The MCA should cover all potential projects from SC2 to SC4. Dependent on the structure category the detailed structural design may require additional data and confirmation/correction of the anticipated morphological development in the identified area. However, despite of all precautions and prediction tools it must always be considered that the dynamic behaviour of the major rivers of Bangladesh occasionally demands drastic changes of scheduled implementation plans.

After confirmation of the proposed structure by the superior Water Board Authorities subsequent to the MCA, the further planning and preparation for the structure implementation has to be initiated. The individual planning processes and the related working steps during the project realization, as summarized in Fig. 3.6-1, are presented in detail in Chapters 4 to 9 of the Guidelines and Manual.

3.6.2 Ranking of Projects

Because of the restricted available funds and the large number of active rivers in Bangladesh, a ranking of the different potential site locations for implementation of erosion counter measures is essential to allow for a sustainable use of the financial resources. It is apparent, that a weighing between different potential projects requires a planning stage up to the conceptual design level. The essential basic information must be provided primarily by the regional/subregional Water Board Authorities, which should also become responsible for the preliminary concept of the envisaged structures. However, due to the specific character of the SIPPS, the key decisions should be taken during the MCA at a superior level of the Water Board Authorities, also with regard to the available respectively required funds. The general steps and the required information to arrive at a priority ranking for the most urgent measures is schematized in Fig. 3.6-2.

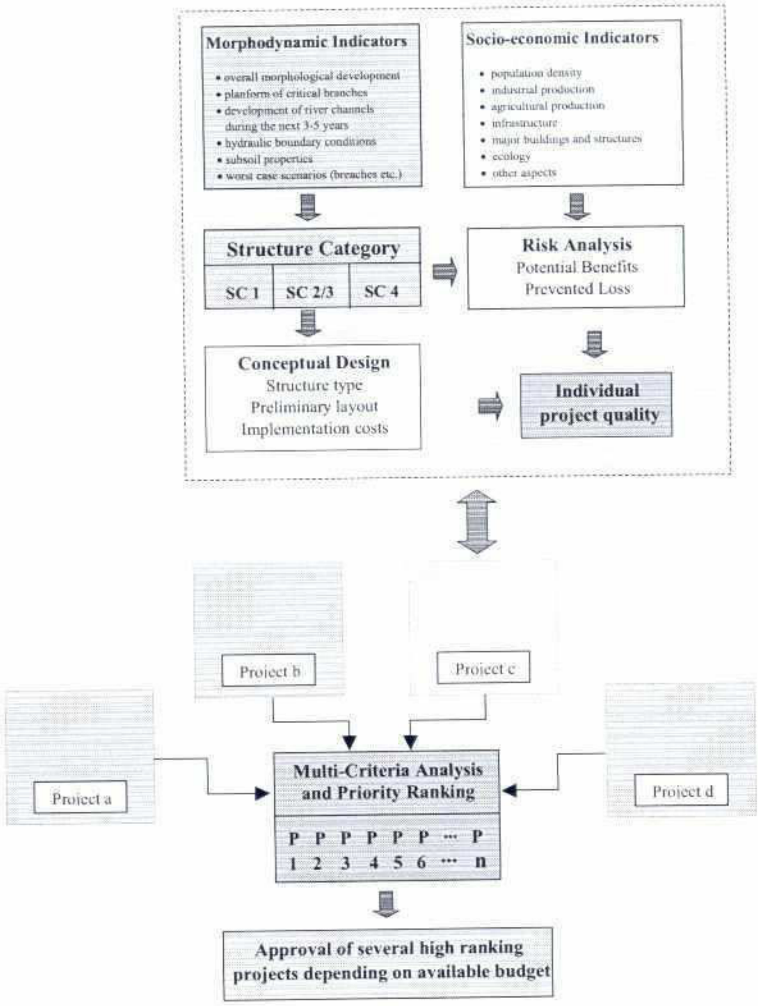


Fig. 3.6-2: Multi-criteria approach and priority ranking of proposed individual bank protection projects

3.6.3 Financial/ Economical Viability Aspects

Engineering efforts (planning, construction) are always associated with considerable capital investment and, on the other hand, any investment is projected to achieve a certain financial return. The outcome is based on a weighed analysis of both components within a feasibility study. In case of protection measures the motivation is based on much wider considerations, in particular if human lives are threatened, or their basic living essentials are at risk. This will increase the complexity of a feasibility study but should be also considered by the decision makers.

Instead of the evaluation of possible returns against the capital investment, considering market uncertainties and risks for profit-oriented activities, the evaluation of protective measures requires the introduction of 'prevented losses'. These are losses which would potentially occur, if a projected measure is not carried out ('without case'). In addition to the risks and uncertainties of the market, which are inherent in the prevented loss, the uncertainties in estimating the probability of actual future damages are apparent.

The statistical analysis of occurrence probability is usually described by theoretical frequency distributions, resulting in a fictitious return period for certain event conditions endangering a specific location. Uncertainties in prediction of natural hazards like earthquakes, floods, etc. are increasing with increasing fictitious return periods and also the precise time of occurrence is not known. In this context, it has to be kept in mind, that statistical analysis only allows a restricted extrapolation to larger return periods (normally twice the time period considered in the data records). The total maximum value of a certain boundary condition is not known, but in general, natural processes tend to follow a saturation curve (asymptotic approach of a constant value), i.e., looking at longer return periods, the extreme forcing boundary conditions are relatively narrow banded. For example, maximum water levels in significant alluvial river are mainly influenced by the precipitation in the vast catch area which levels out most extreme events by combining the respective return periods of several zones in the overall catch area. Moreover, if the bank full discharge is exceeded, the further increase in water level is limited. Hence the difference between the maximum water level according to a return period of 50 years and 100 year is relatively small.

As aforementioned, the estimation of return periods requires a substantial data base for the particular region or site location which is consisting of parameters describing the temporal properties (frequency, duration) in relation to the intensity of a specific type of hazardous event. This information is not available for natural phenomenon and physical processes, (i) which occur too infrequently (e.g. earthquakes in the Bangladesh region), (ii) which are subjected to a large number of influencing parameters or which has a strong spatial dependency (local effects). Therefore, the definition of a return period in terms of bankline erosion for high mobility rivers is not possible, because the existence and the properties of major influential channels as well as their further development and interference at a distinct location cannot be based on a sufficient data set. For the future it is expected that, due to intensive monitoring and improved modelling techniques, reliable prediction methods are available which will also contribute to the assessment of potential prevented losses over the structure life time.

However, especially when considering bank erosion measures, the main focus should be pointed on comparability of different projects, using the same rules and basic assumptions for the estimation of the future development, for which different pre-defined scenarios should be used. A simplified approach would be the evaluation and extrapolation of measured erosion rates over the past years prior to the intended project realization. For the matter of comparability, the annual fictitious eroded area can be calculated by the annual erosion rate and the protected length of the bank line. From both, the upstream and downstream terminations a 45° line could be assumed to take into account the indirect positive effect of the structure (Fig. 3.6-3).

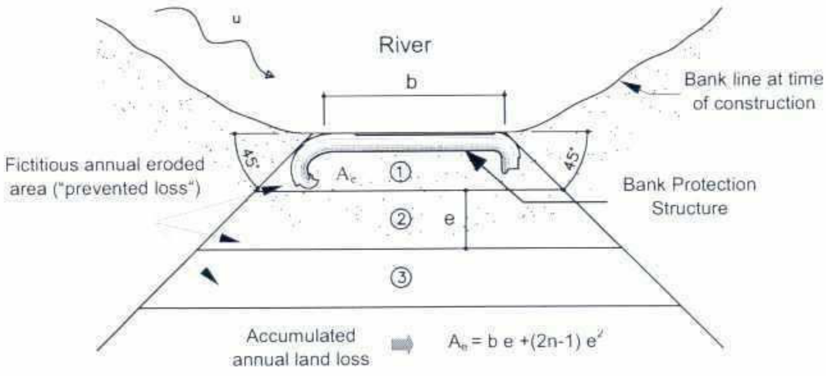


Fig. 3.6-3: Simplified approach to account for prevented losses in the feasibility study (scheme)

Combined with the existing socio-economic indicators (population, infrastructure, buildings, etc.) the fictitious prevented annual losses and the recurrent accumulating losses (land use, cropping pattern, productivity, etc.) over the projected structure life can be assessed.

On the cost side, all capital investment required for implementation of bank protection measures (e.g., land acquisition, compensation, construction material, consulting) as well as estimated recurrent costs for future maintenance and repair works have to be considered.

Remaining uncertainties regarding the estimation of costs and benefits should be assessed in a sensitivity analysis, in which the most significant but only roughly predictable input parameters are varied over a reasonable range.

To allow for a comparison between different feasibility studies (various potential bank protection projects), the cost.-benefit-analysis must follow defined standards, which should be identified in collaboration between river engineers and economists. Assistance in this context can be found in publications of the European Communities (1997), the Asian Development Bank (1997) and the FPCO Guidelines (1992).

3.7 PROJECT REALIZATION

3.7.1 General Implementation Schedule

Once all pre-investigations are completed and a site has been identified through a MCA (refer to Fig. 3.6-1) as a Priority Project, the **P**roject **P**lanning **S**chedule (PPS) would outline the individual tasks and milestones to be accomplished and the total time required for implementing a specific project. The PPS shall be the guideline for the implementing authority to comply with the administrative and legal matters on time and to monitor the work progress of their consultants (if any) and the contractor(s) to ensure that the project will be implemented within the desired period of time.

Dependent on the nature and magnitude of a project, individual tasks may be of different importance along the critical path of the project. Among others the following subjects are of relevance for drawing up the PPS:

- time to secure project financing
- the construction window, which is pre-determined by the river's hydrograph;
- type of structure, i.e. "Standardized Structure (SC 2 or SC 3)" or "Special Design (SC 4)";
- time for preliminary and detailed design;
- tendering procedure through "Local Competitive Bidding" (LCB) or "International Competitive Bidding" (ICB);
- time for procurement of materials and equipment, and
- land acquisition.

Any PPS has to start from the assumption that construction works must start as early as possible after a monsoon season to maximise the construction window. Legal and financing issues as well as all design, pre-qualification of contractors, tendering and contract award procedures must, therefore, be completed before the end of the monsoon season.

A "Fast Track Method" can be employed for projects falling under category SC 2 and SC 3 to take full advantage of the simplified planning processes. Thus, such project can be implemented within much shorter time as compared to the full-scale implementation processes for projects under category SC 4 (Fig. 3.7-1).

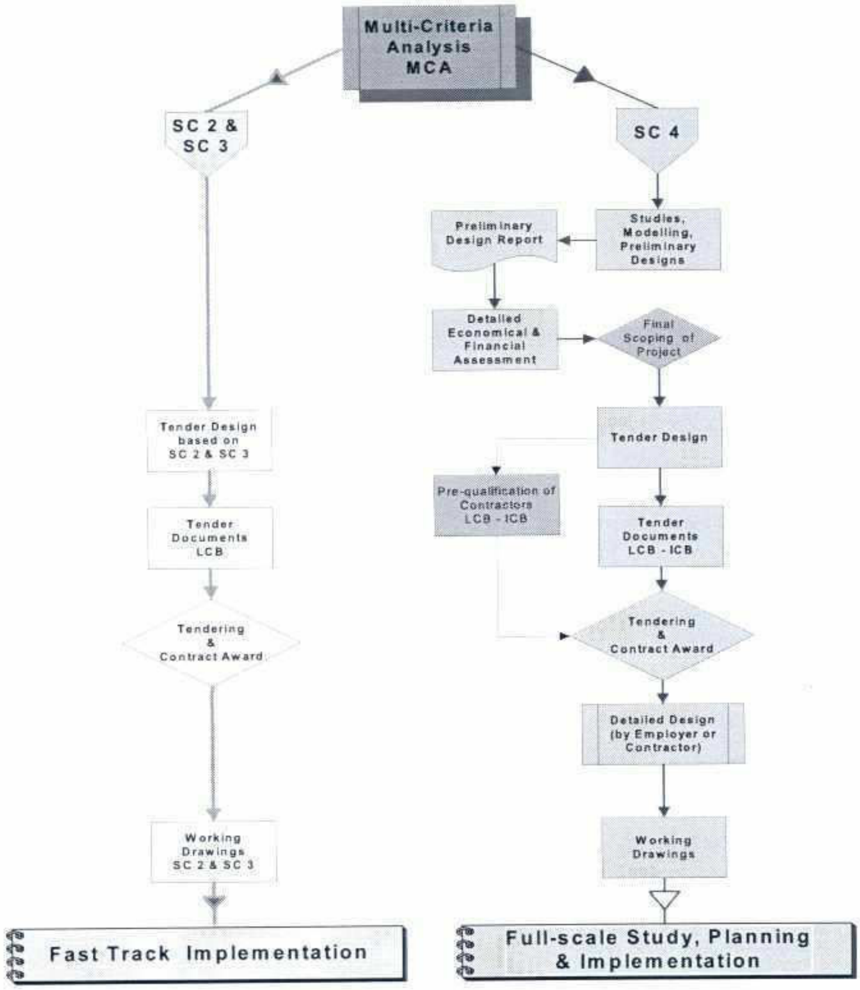


Fig. 3.7-1: General planning and tendering process

3.7.2 Preliminary Designs

Irrespective of the Structure Category any preliminary design shall include basic considerations of hydraulic and structural loads and stability calculations. The designs shall be presented on drawings suitably scaled to clearly demonstrate the planned measures. Quantity surveys and order-of-magnitude cost estimates shall accompany each preliminary design. The results of such preliminary design should be compiled and presented in a Preliminary Design Report (PDR). If more than only one technical option may be feasible, the alternatives should be compared in a matrix of criteria to be defined in consideration of the project specific requirements. Such selection criteria may, among others, include the following criteria:

- applicability of Standardized Designs;
- anticipated river response;
- requirement of supplementary measures in the years after construction;
- material availability from Strategic Stocks;
- advance procurement of materials;
- import of materials;
- suitability for local construction capacities;
- tendering requirements (LCB/ICB);
- investment cost;
- anticipated maintenance cost during first three years after construction;
- anticipated cost for supplementary measures in the years after construction;
- acceptance by local population;
- land acquisition area, cost of land and compensation, and
- overall implementation time.

The PDR shall conclude with a recommendation for the most feasible design solution for the respective project. These inputs may be introduced to the MCA (refer to Fig. 3.7-1) enabling the implementing authority to decide and to go (or not to go) ahead with the project.

3.7.3 Tender Design

The basis of the Tender Design shall be the approved PDR. In case of a considerable time gap between completion of the PDR and the preparation of the Tender Design, e.g. one complete monsoon season or even more, the basic planning and design parameters have to be verified and, if required, to be adapted well in consideration of the most recent morphological and otherwise related changes at the selected project site.

The Tender Design for SC 4 type projects shall be based on the detailed pre-investigations, as well as on structural and stability computations for the main elements of the structural measures. Contrary to this, a Tender Design for a SC 2 and SC 3 type project may be based on the standardized structure elements recommended in the Manual. The tender design drawings shall depict the structures and all essential details so that any bidder can clearly identify the appropriate construction methods, the material requirements, the constraints and risk involved with the work execution and subsequent maintenance.

The tender design drawings will become an integral part of the Tender Documents referred to under Section 3.7.4. The design assumptions as well as material and executional requirements shall be compiled and included in the Technical Specifications of the Tender Documents for the information of the bidders and the contractor respectively.

3.7.4 Tender Procedures

3.7.4.1 Introduction

Dependent on the magnitude of the respective project, whether by value or demanded construction capability and experience, or due to financing from local budget or international funding, a project may be required to follow LCB (Section 3.7.4.2) or ICB (Section 3.7.4.3) procedures. The implementing authority has to decide on the desired type of the bidding procedure.

Usually Tender Documents for SC 2 and SC 3 projects may be based on a Tender Design prepared by the implementing authority, but tender designs for more complex and full-scale projects falling under category SC 4 may be prepared with the assistance of or by a qualified consulting engineering firm on behalf of the Employer.

In recent time, however, employers tend to delegate the design responsibility to the contractor carrying out the construction works under Design-Build or Engineering - Procurement – Construct (EPC) procedures. Though this is more popular for major civil engineering structures, such as bridges, quay walls, building structures, etc., it may not be ruled out for the SC 4 type of projects discussed in these Guidelines. For Design-Build or EPC procedures the implementing authority would provide only a conceptual design along with the basic assumptions that the respective structure must meet after completion (the Employer's Requirements). Within these Guidelines, however, it is being assumed that the implementing authority will provide the design to the respective contractor. Reference to other methods is only provided to acquaint the reader with recent developments.

3.7.4.2 LCB – Tender Documents

For SC 2 and SC 3 projects the tendering should follow Local Competitive Bidding (LCB), as per standard procedure of the BWDB for large projects (LCB-2 for construction volume > 25 lakh).

This document covers all conditions of LCB contracts both for GoB and Donor financed projects/works. Provisions as per ERD guidelines for procurement and relevant conditions of the funding partner(s) are also included in the document. It usually comprises:

- General Conditions of Contract;
- Special Conditions of Contract;
- Technical Specifications, and
- Tender Drawings.

For larger projects, i.e. those falling under category SC 4, it may be appropriate to enhance the presently applied procedures by application of the elements for ICB, with a view to attract foreign firms to participate in the bidding for such projects.

The Technical Specifications for any construction work under LCB should be based on the latest edition of BWDB standard document "Technical Specifications for Civil Works". This

97

standard document may be supplemented to also cover more recent methods of erosion protection, such as articulating mattresses, geo-synthetic mattress systems, etc.

3.7.4.3 ICB – Tender Documents

Tender Documents for International Competitive Bidding (ICB) may be based on conditions and regulations acceptable to the international community of employers, financing agencies, consultants and contractors. The tendering procedures issued by the International Federation of Consulting Engineers (FIDIC[1]) have gained widespread acceptance and are suggested for application also for ICB procedures in Bangladesh. In case of foreign funded projects some particular conditions of the donor agency may have to be considered.

Basically ICB tender documents shall comprise the following elements and it is suggested to maintain the arrangement of volumes, which has proven its practicality:

Volume I Form of Tender and Bid Security
Instructions to Tenderers
Sample Form of Agreement
Sample Forms of Bank Guarantees
Bill of Quantities
Schedules
Volume II General Conditions of Contract
Conditions of Particular Application
Volume III Technical Specifications
Volume IV Tender Drawings

Volume I is intended to include all those schedules, lists and forms that are to be completed and signed by the tenderer. For this purpose the Tender Documents issued by the Employer may include one set of Volume I unbound, enabling the bidders to complete the pages as required, to reproduce the number of copies to be submitted and to compile and bind Volume I for bid submission. The bills of quantities may be provided to the tenderers in digital spreadsheet format, enabling them to insert the unit rates and to generate the amounts and totals of the bills and to print out the completed bills. In this case, however, the entire document of the bills should be password protected, except the spaces where rates are to be inserted.

The Instructions to Tenderers are to stipulate clearly the requirements to be complied with by the tenderers in preparing the bid. This includes information about bidder's eligibility criteria to be fulfilled, compulsory pre-bid meeting, the format in which the tender is to be submitted and the number of copies to be provided, the date, time and place where the tender is to be submitted (the due date), regulations for handling bids received after the due date, information on the bid evaluation and contract award procedures, etc.

For major projects, i.e. those falling under category SC 4, it should be made compulsory for tenderers to participate in a pre-bid meeting. The implementing authority (the Employer) may

[1] Fédération Internationale des Ingénieurs-Conseils, Lausanne/Switzerland

arrange such meeting with the assistance of the consulting engineering firm (the Engineer) hired by the Employer for the design and tendering of such project. The pre-bid meeting should preferably include a joint site visit with an explanation of the aim, features and merits of the project, followed by a question and answer session. Only where the project site is extremely remote, the pre-bid meeting may be held at the Employer's premises, but each bidder would be required to visit the site at his own arrangement and expense.

The Instructions to Tenderers shall also include specific sample forms for the tender submission, bid security (bid bond), the agreement with forms for performance bond, advance payment guarantee, etc., to ensure complete information to the bidders on all administrative, legal and technical requirements.

The Bill of Quantities shall comprise a preamble, the summary sheet, the individual bills, daywork schedules and schedules specifying submittals to be provided by the bidder. The preamble shall clearly specify the requirements for bidding the rates and prices and for completing the bills. Among others it must be well defined which cost components are to be included in site installation cost (namely only site and equipment mobilisation cost) and which ones in the work items (namely, among others, the running cost of equipment), etc. If clear conditions are not set out for pricing, the analysis and comparison of unit rates and prices in the process of bid evaluation will become speculative, if not impossible.

The quantities shown in the bills shall be based on reasonable estimates; hereby the final measurement of the volume of the completed works may be used for settlement of accounts. Lump sum items should only be used, if the respective item or work can be described clearly without any hidden aspects.

The Daywork Schedules shall include a standard set of man-hours of different skills, various construction materials as well as construction equipment, all appropriate to the project in question. The total of the daywork schedules should become a part of the total of bills, i.e. they will become an element of the tender price. Thus, daywork rates are offered under competitive conditions and it can be avoided that inappropriately high daywork rates would lead to unbalanced cost for any unforeseen work item.

The Schedules of Supplementary Information to be attached to the Bill of Quantities shall specify the additional information to be provided by the bidder along with the bid. The bidder's failure to comply with this requirement would lead to disregarding the tender. These schedules should, among other things, refer to the following:

- proposed head office management and site organisation;
- list of leading personnel, including brief CVs;
- information about intended suppliers and sub-contractors;
- detailed work method statements for selected construction works;
- description of construction stages;
- basic design of major temporary work items;
- construction time schedule;
- material procurement schedule;

18

- equipment deployment schedule, and
- quality assurance programme.

Volume II comprises the administrative-legal part of the Tender Documents and may be drafted for one of the two conditions

A. The implementing agency (the Employer) provides the Tender Design as well as the Detailed Design to the Contractor, and the Employer (or the Engineer hired by the Employer) will be responsible for the adequacy of the design. In this case the Contractor will be responsible only to prepare the shop drawings, to design any temporary work and for the supply and construction, including rectification of executional deficiencies during the Defects Liability Period agreed for the works,

or

B. The implementing agency (the Employer) provides only a conceptual design and specific parameters and requirements (the Employer's Requirements), but the Contractor has to carry out the detailed design, supply, construction as well as maintenance during the Defects Liability Period under his sole responsibility.

Dependent on the intent of the implementing agency, the General Conditions of Contract and supplementing Conditions of Particular Application may be selected from

A. **Conditions of Contract for Construction**
(for building and engineering works designed by the Employer)
First Edition 1999 (Red Book)
ISBN 2-88432-022-9
© Copyright FIDIC 1999

or

B. **Conditions of Contract for Plant and Design-Build**
(for electrical and mechanical plant, and for building and engineering works, designed by the Contractor)
First Edition (1999) (Orange Book)
ISBN 2-88432-023-7
© Copyright FIDIC 1999

Alternatively:
Conditions of Contract for EPC Turnkey Projects
First Edition (1999)
ISBN 2-88432-021-0
© Copyright FIDIC 1999

For construction works of medium size projects of category SC 4, or in special cases even for projects with SC 2 or SC 3 type structures but which shall nevertheless be subject to ICB procedures, a short form of contract may be used (note that this procedure may not meet always with donor's with requirements). The standard document to be used has likewise been developed and published by FIDIC, and can be adapted to apply to both options A or B as detailed above:

C. **Short Form of Contract**
First Edition (1999)
ISBN 2-88432-024-5
© Copyright FIDIC 1999

Volume III comprises the Technical Specifications and shall describe the

- project location and scope of works
- local conditions, including subsoil, onshore and offshore conditions, site access, etc.
- detailed description of works
- planning and design requirements (Employer's Requirements where appropriate)
- standards to be applied
- documents to be provided by the Employer
- documents to be provided by the Contractor
- material tests and workmanship
- setting-out and survey
- specifications for each category of work and/or supply
- methods of measurement of the works
- quality control and performance records

and shall be the fully comprehensive technical contract document to be used throughout the construction works and the defects liability period. Any supply and construction work shall be measured against the criteria and parameters set out in the Technical Specifications to enable the Employer or the Engineer either to approve or reject the Works or parts thereof.

Volume IV compiles the Tender Drawings, illustrating the kind of project and type of construction to be carried out. Tender Drawings (for that matter also any other detailed design and/or working drawing) are always to be read and interpreted in conjunction with the Technical Specifications and the Bill of Quantities.

As a matter of principle Tender Drawings should include, among others

- location map;
- limits of site area and its approaches;
- layout plan with project co-ordinates for setting-out;
- subsoil information (exploratory borings, CPT results, grain size distribution, etc.);
- dredging/earthworks/filling/reclamation plans;
- detailed structure/facilities layouts;
- detailed sections of the structures/facilities;

62

- typical details of structures and facilities;
- typical reinforcement drawings for main structural elements, and
- general construction time schedule.

The original size of Tender Drawings, as well as any detailed design and/or working drawing, should be handy enough for use at site, for which drawing size DIN A 1 (594 x 841mm) has proven to be suitable. The Tender Drawings, however, may be plotted to size DIN A 3 (297 x 420 mm) and bound in a proper A 3-size folder to become Volume IV of the Tender Documents.

3.7.5 Selection of Construction Methods

For structures of SC 2 and SC 3 typical and proven construction methods and equipment deployment will be standardized to some extent. Nevertheless, for any construction the contractor who has been awarded the contract for execution of the works is solely responsibility for the selection of the appropriate construction method(s). Within this context the contractor has to decide on and is responsible for the method and sequence of the work execution and has to provide the type and capacity of equipment to carry out the works in accordance with the Technical Specifications and the Drawings.

Already at the preliminary design stage of a project the selection of basic construction methods is an interactive scenario with the study of design options for the structure(s) in question. The most feasible design can only be arrived at, if the likely method of construction is being pre-determined, well in consideration of the particular local conditions (including subsoil) and the applicability of the design to the use of local construction capability and experience.

Particular project constraints may emerge for SC 4 type projects as these are likely to be executed under severe conditions, such as high river flow, excessive water depth, or with substantial volumes of works to be carried out and completed within just one dry season construction window. Such conditions would call for application of enhanced constructions methods and utilisation of specialised and/or high equipment capacity, which may not (yet) be available within Bangladesh. Such circumstances would typically call for ICB procedures as referred to under Section 3.7.4.3.

3.7.6 Procurement of Construction Materials

Procurement of construction materials should be based on strategic material stockpiling at selected locations. This will enhance the fast track implementation of standardized structures, and would ensure that the construction works can commence as soon as possible after a monsoon season.

In any case, the timely procurement of construction materials is of eminent importance and the success of project completion within the available construction window will substantially depend on it. The following should be given due consideration when planning the project implementation:

- Owing to the fact that (due to aftermath of monsoon season) new brick productions would be available only after early December the respective works would start too late and valuable production and construction time is lost. Therefore, it is a good practice to procure bricks for mattresses and concrete block production well in advance, i.e. from previous productions, at least to a quantity sufficient to cover the period until new supplies are available.
- Should the project design require substantial quantities of hard rock, boulders and stone material, which could be a cost effective alternative to the use of concrete blocks, early procurement is likewise important since these materials are usually imported from neighbouring countries. The collection of substantial material volumes from the natural sources and the subsequent selection, blending and transportation is time consuming.
- Permeable groyne structures of prefabricated piles, such as tubular steel piles or factory-made reinforced concrete piles require a considerable lead-time for material procurement. Though it is considered that tubular steel piles of the desired standard diameters and lengths can be produced in Bangladesh in future years, a lead-time of 6 to 8 months must be taken into consideration for the latest start of such pile production. Factory-made reinforced concrete piles are available from a factory in Bangladesh on special order and can be produced well ahead of construction start. It should be kept in mind, however, that such concrete piles have a limited application range.
- Geo-textile filter materials have to be imported though it should not be ruled out that in future years local manufacturing of two standard types of geo-textile filter materials may be feasible. These materials are available from a wide range of manufacturers in Asia and Europe and are commonly available for shipment within a short period of time. Allowing reasonable time for shipment to Bangladesh, clearance and transport to the site, a lead-time of about 3 to 5 months should be considered. Again, a strategic material stockpiling would permit to take over such materials from the material stocks kept under the custody of BWDB.
- Geo-textile mattress systems have to be imported. The production requires sophisticated waving machinery for which any future local production may be ruled out. These mattress systems are only produced on special order and the complicated waving procedure requires a considerable production time. A lead-time of 6 to 8 months should be considered in the procurement schedule of the respective project.

3.7.7 Prequalification of Contractors

3.7.7.1 Introduction

It is a well-accepted practice to launch pre-qualification (PQ) procedures to select competent bidders to participate in tenders for civil works contracts. Through these procedures it shall be ensured that contracts are only awarded to bidders, which are qualified having the appropriate capabilities, experience and financial resources.

The procedures to be applied for pre-qualification of capable civil works contractors depend on various parameters interconnected with the chosen bidding procedure, i.e. LCB (Section

3.7.4.2) and ICB (Section 3.7.4.3). Therefore, the following subsections differentiate between PQ procedures for local as well as international competitive bidding.

3.7.7.2 PQ-Procedures for LCB Projects

Pre-qualification of contractors is usually done by BWDB for large or complex works on the basis of detailed estimates to ensure in advance of bidding, that invitations to bid are extended only to capable bidders. Bank protection projects would normally suit so-called Class A contractors registered with the authorities. With a view to the recommended standardized structural measures for SC 2 and SC 3 it may be considered for the future to introduce a structure specific classification of potential bidders.

3.7.7.3 PQ-Procedures for ICB Projects

Different procedures have been developed and utilised in the past for prequalification of civil works contractors. Though the form may be different, they all have common elements, which are

- Instructions to the applicants identifying the type of works, the source of financing and the general requirement to be complied with by the applicant
- Short description of the project country, the contract site, a description of the works, including quantities of major work components and basic construction methods required, thus to permit the applicants to understand the project requirements and to assess whether or not such requirements could be met by the applicant
- A set of purpose-made forms to be filled and completed by an applicant, among others to compile all company related data, the financial status, joint venture data (if applicable), the firms' experience with similar works and under similar conditions, the summary of project key personnel, equipment proposed for the project, proposed site organisation and subcontractors, etc.
- Information about the stage-wise assessment of the applications
- Description of the major features of the scoring system, which may include
    - groups of evaluation factors, such as financial capacity, technical capacity and experience
    - criteria for assigning numerical values to the quantitative information submitted by the applicants
    - group factors to best reflect the relative importance of the respective group for the determination of an applicant's overall qualifications

Dependent on the financing source of a project the funding agency may have specific requirements to be followed in the PQ-procedures. These may include the format of invitation, the use of specific channels/media and publications to announce the pre-qualification, adherence to pre-determined minimum periods between announcement and dead line for document submission, inclusion of specific eligibility criteria to be fulfilled by the applicant, etc..

Among the available recommendations and guidelines for prequalification of contractors the "Tendering Procedures" issued by FIDIC and the "Guide on Prequalification of Civil Works Contractors" published by the Asian Development Bank (ADB) are useful examples and may be considered when drafting purpose-made PQ procedures and documents.

3.7.8 Detailed Design

It is being assumed that for SC 2 and SC 3 type projects the detailed design will be carried out by the implementing authority, while detailed design for SC 4 type projects may be carried out with the assistance of a qualified consulting engineering firm. The detailed design shall further detail and supplement the Tender Drawings, taking into consideration most recent changes at the finally selected and confirmed construction site. For SC 4 projects the detailed design shall include all final computations and stability analyses and the drawings shall show all information and dimensions needed by the Contractor to prepare the working drawings, bending schedules and shop drawings.

Working drawings for SC 4 projects shall supplement the Detailed Design referred to under Section 3.7.8 and may have to be prepared either by the employed consultant (the Engineer) or the contractor, which depends on the scope of consulting engineering services awarded to the Engineer by the Employer.

It is a good practice, however, that the Contractor prepares all working drawings, including bar bending schedules, shop drawings, etc., while the Engineer shall check Contractor's submittals for conformity with the contract specifications and the otherwise agreed (Employer's) requirements. The Contractor shall only carry out the respective construction works when the Engineer has approved the related working drawing.

3.7.9 Land Acquisition

3.7.9.1 Approach to the Work

Possession of land for the construction is a prerequisite in all project works. This is achieved by acquisition of land through government procedure. The procedure comprises the following:

(a) preparation of the site plan encompassing the area permanently required by the project;
(b) collection of relevant mouza maps and transferring the site plan into the mouza maps;
(c) preparation of schedule of land falling within the site plan drawn on the mouza map with exact area and ownership;
(d) preparation of the inventory of affected households covering mainly information about housing (type, size, materials used etc.) and other items (tube wells, latrines etc.). Information about shops, business units, rural industry as well as trees, standing crops in the field, ponds and fish population in the ponds etc. are also needed to be included in the inventory;
(e) preparation of the land acquisition proposal in the required format based on information in the above land schedule and inventory, and

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(f) submission of the proposal to the Deputy Commissioner for formal acquisition of land.

The Deputy Commissioner's office needs about 5 to 6 months for stepwise notifications to the people having property in the proposed acquisition plan and makes verification of inventory of houses, crops, trees and others. An estimate is then prepared for the compensation as per approved rates of the items included in the acquisition proposal for payment by the requiring body. After obtaining the fund from the requiring body, the Deputy commissioner's land acquisition office acquires the land on payment to the affected people and hands over the same to the requiring body.

3.7.9.2 The Problem and Formulation for Solution

Bank protection works are very much related to the bankline at the time of construction. The necessary preparation for all other works like preparation of design, floating of tenders and procurement of materials can be done well ahead for a site area with the probable bankline at the time of construction keeping in mind the erosion rate prevalent at the site. But the exact location of the structure can not be fixed until the recession of the flood, hence, unlike other projects, not well ahead but only a couple of weeks before the start of the actual construction works. This peculiarity of bank protection projects with respect to land acquisition calls for some formulation to solve the problem.

The formulation consists of two aspects of activities. The first aspect is to prepare umbrella coverage of the probable site of construction and follow the steps (a) through (f) of the approach to the work (see Section 3.7.9.1). This can be done immediately after the project is taken in hand for implementation.

The second aspect of formulation deals with the situation of making the land available for the construction in the very beginning of the construction window in November with the target of completion of the project in May or June of the following year. The work approach for making the land available in time comprises:

(a) fix the alignment of the structure in exact location after recession of the flood in the month of September along the actual bankline in the umbrella coverage of the probable site and prepare the land plan and land schedule as per modified location map;
(b) hold participatory public meetings to discuss thoroughly the pros and cons of the project and convince the likely affected population and beneficiaries about the necessity of the project;
(c) if the population agrees with the implementation of the project and assures necessary help and assistance in the project work, emphasis may be given that land should be made available to the project before the actual payment of compensation for land by the appropriate authority. This can be achieved by immediate payment borne directly by the project as per mutually agreed rates on cost of house shifting and compensation for standing crops and trees. Funds should be allocated by the project for this purpose.

While the first formulation will proceed and make land permanently acquired in due time, the second formulation will make land available on temporary basis.

3.7.9.3 Fund for Land Acquisition

Projects taken in hand for protective works do not allow much lead time and land acquisition is imperative for the implementation of the project. Therefore, sufficient fund needs to be kept for payment of land compensation and the whole allocation in this respect of the annual development plan budget should be made available at the beginning of the second quarter of the fiscal year.

4 HYDRAULIC AND MORPHOLOGICAL BOUNDARY CONDITIONS

4.1 INTRODUCTION

The banks along the braided Bangladeshi rivers are eroded frequently as the planform changes are usually very dynamic. The natural erosion of unprotected banks as the main failure phenomenon should be understood well before defining hydraulic and morphological boundary conditions for the design. Reference is made to Chapter 2 with a review on the processes of erosion and mass failure of banks. These failure modes of bank erosion yield the requirement for specific hydraulic and morphological design parameters.

The driving forces for erosion are the high shear stresses by current flow, which exceed the shear strength of the soil. The gradual process of surface erosion can trigger a sudden mass failure of banks. Different failure modes are anticipated to occur at a critical slope of the river bank. Thus, bed erosion at the toe in vertical direction will cause bank erosion and a self induced adaptation to a slope which is milder than the critical one. For the design of a protection structure, also the importance of scouring is evident and consequently has to be taken into account in the design phase. The size of local scour holes and their development with time has to be estimated after a choice of the conceptual approach for the protection structure has been made.

General design parameters, that are not dependent on the type of protection structure, form a basis for the individual structural design. The hydraulic design parameters, e.g. design water level and design flow depth, act as boundary conditions for the morphological parameters, i.e. the design cross-section of the river. The design flow velocities are not directly used to evaluate the risk of bank erosion. In this context, scouring to a certain scour depth is the decisive phenomenon and consequently the main parameter for the design. However, an estimation of the flow velocities is needed to optimise the construction phase of the protection work, with respect to the placing of materials and constructional elements.

In the following, subsequent working steps and simplified approaches for the estimation of input parameters for the structural design are explained. Details on the hydraulic design parameters are given in Section 4.2. The determination of the design cross-section is treated in Section 4.3 and 4.4. Section 4.5 covers the determination of design flow velocities and Section 4.6 comments on other boundary conditions for the design.

4.2 HYDRAULIC DESIGN PARAMETERS

4.2.1 General Approach

River training structures are generally designed to resist a certain design flood. In Bangladesh, a flood with a recurrence period of 100 years is commonly selected as the design flood for bank protections of primary importance. The lifetime of a protection structure has often been related to the recurrence period of the design flood. Therefore, it is assumed that the design will be based on a lifetime of about 50 to 100 years for a bank protection structure. The service time of a river training structure is only a fraction of the lifetime, because the structure experiences river attacks only in a limited number of years, lying idle in other years. Thus, design conditions with a recurrence period of 100 years have a much lower probability of occurring during the service time, which implies that the common approach leads to over design.

On the other hand, the most severe loads on the structure are not related to the most extreme flood events, so that hydraulic recurrence periods derived from water level registrations or discharge registrations are not very representative for the design conditions. It is likely that extreme planforms will have a significant influence on the design parameters, but this influence is still relatively unknown and has to be estimated. A morphological recurrence period of the same safety level as the hydraulic recurrence period can only be applied if sufficient data are available (see Section 4.3).

Therefore, two approaches will be followed for the design:

- The relatively long hydraulic recurrence period of a design flood will be maintained;
- Extreme hydraulic loads due to planform development will be considered by an adequate choice of the design cross-section, where consideration is given to extra structure induced scour and extreme oblique flow attack.

The discharge of the design flood is defined in order to estimate the Design Water Level (DWL) with a return period of 100 years. Additionally, a bankfull discharge is defined for the estimation of the design cross-section. The bankfull flow conditions are also representative for the morphological situation of the riverbed during flood conditions, as the difference in water level is small and an increase of sediment transport from bankfull to flood flow conditions negligible.

4.2.2 Design Discharge

The discharge of a specific river is obtained from the analysis of hydrological data, especially through extrapolation of stage-discharge relations at water level stations, where also discharge measurements have been executed regularly. The analysis of flood discharges and the associated recurrence periods result in a probability function which can be used to define the design discharge with a return period of 100 years.

If no sufficient data on the discharge recurrence period are available, the total discharge in the river with a return period of 100 years can be approximated at a value of about twice the bankfull discharge. In a braided river, the total discharge is distributed over several channels and will partially inundate the floodplain.

The design discharge Q_{ch} for a channel can be described by:

$$Q_{ch} = \frac{c_1}{c_b} \cdot 2 \cdot Q_b \qquad \text{(Eq. 4.2-1)}$$

with

Q_b (m^3/s) bankfull river discharge
c_l (-) safety factor for extreme channel discharge, $c_l = 1.5$
c_b (-) braiding index

The braiding index c_b represents the number of channels and lies between 4 and 5 for the Jamuna river. For the Ganges and the Padma $c_b = 1$ should be used, as they are not braided. For other rivers of Bangladesh, it is recommended to estimate c_b from satellite images. The bankfull discharge Q_b can be taken from Table 4.2-1.

River	Bankfull discharge (m^3/s)	Return period of bankfull discharge (years)	Braiding index (-)
Jamuna	48 000	1.00	4 - 5
Ganges	43 000	1.40	1
Padma	75 000	1.05	1

Table 4.2-1: Discharge characteristics for major rivers of Bangladesh

The bankfull channel discharge $Q_{b,ch}$ is calculated by:

$$Q_{b,ch} = \frac{Q_b}{c_b} \qquad \text{(Eq. 4.2-2)}$$

with

Q_b (m^3/s) bankfull river discharge
c_b (-) braiding index

4.2.3 Design Water Level

The Design Water Level (DWL) is related to PWD and can be derived from the design discharge. As the relation between discharge and water level varies due to rapid morphological changes, only a stage discharge relation (rating curve) established at the location of the planned structure from long term monitoring of daily averaged water levels and corresponding discharges can be used. For selected locations, data is available from the Surface Water Modelling Centre (SWMC). The SWMC also provides data on the water level of Standard Low Water (SLW).

To calculate the DWL in m+PWD, an extrapolation of the stage discharge relation is to be used. Extrapolation is necessary, as it cannot be expected that the design conditions can be observed during the monitoring phase at the location of the structure prototype. However, from experience it can be stated that the water level does not change much between recurrence periods of 25 or 200 years. The DWL is generally above the Flood Plain Level

(FPL), the return period of bankfull discharge is much shorter (for Jamuna, Ganges and Padma between 1.0 and 1.4 years, see Table 4.2-1).

4.2.4 Waves

In general the maximum wave heights are caused by maximum wind speed over the longest fetch length. From observations it is concluded that the maximum wave height with a return period of 100 years for the design of bank protection structures along the Brahmaputra-Jamuna River should be about 1.3 m. For a return period of 25 years a design wave height of 1.0 m is commonly used for major rivers of Bangladesh.

However, an approximate calculation of the wind induced wave heights can be done by theoretical methods in order to verify these assumptions and to investigate the band width of results. The design method is based on the formulas proposed in the Shore Protection Manual (CERC, 1977). For shallow water conditions, present during the continuous changes between sand bars, chars and tributaries, the significant wave height Hs is calculated by:

$$H_s = 0.283 \cdot \tanh\left[0.530 \cdot \left(\frac{g \cdot h_{ch}}{u_w^2}\right)^{0.75}\right] \cdot \tanh\left\{\frac{0.0125 \cdot \left(\frac{g \cdot L_f}{u_w^2}\right)^{0.42}}{\tanh\left[0.530 \cdot \left(\frac{g \cdot h_{ch}}{u_w^2}\right)^{0.75}\right]}\right\} \cdot \frac{u_w^2}{g} \qquad \text{(Eq. 4.2-3)}$$

in which

h_{ch}	(m)	average water depth in upwind area of water body
L_f	(m)	fetch length
g	(m/s^2)	acceleration due to gravity
H_s	(m)	significant wave height
u_w	(m/s)	average wind speed

Eq. 4.2-3 does not consider effects from wave transformation, strong variance in water depth and the underlying currents. It holds for the deepest parts of the channel. The waves are lower in the shallower areas of the channel. It is recommended to take this phenomenon of shoaling into account when calculating the design wave attack on an embankment. A more reliable theoretical prediction of wave parameters is rather complex and will not be covered here.

Maximum wind speeds mainly occur by wind during the tropical Northwesters and by cyclones. It is recommended to use wind data of the Bangladesh Meteorological Department, Dhaka. The distribution curves for the wind velocity as a function of the return period should be applied for determination of the design wind speed. For the Jamuna areas, a period of 15 minutes as an approximate time for the full growth of wind waves yields an average wind speed of 25 m/s for a return period of 100 years.

The effective fetch length depends on the water level in the channel, the alignment of the channel compared to the wind direction and the banklines of the upwind water body.

Therefore the fetch length can vary widely. It is at its maximum during monsoon, when the river flow is at its peak and many chars are inundated. For the Brahmaputra-Jamuna river, a design fetch length of 2 to 5 km is recommended. Detailed information can be obtained from satellite images.

4.3 DESIGN CROSS-SECTION FROM MORPHOLOGICAL ANALYSIS

For the conceptual planning of a protection structure, a design cross-section of the channel has to be determined. A morphological analysis of data available from SWMC or obtained from monitoring prior to the planning phase serves as the best approach to derive information on the bankfull channel width, the expected water depths and the cross-sectional shape. The bankfull channel width and additionally the bend curvature can also be obtained from satellite images and planform analysis.

Data of the Jamuna River consist of yearly measured standard cross-sections recorded during the lean season. In addition, irregular bathymetric surveys of the ferry routes can be made available. However, to represent the morphological situation of the river bed during flood conditions, cross-sections should be monitored at least at bankfull flow conditions. Morphological changes at flow conditions that correspond to water levels higher than FPL are negligibly small. Even if local erosion of the river bed during flood events produces higher water depths below FPL, in other locations sedimentation will occur so that in the average the cross-sectional profile does not change.

To estimate the outer bank profile and the maximum water depth in the thalweg, it is recommended to use an envelope curve of measured cross-sections. In principle, these sections should not be influenced by any river training structure. It is recommended to determine the statistical distribution of the enveloping curve and to select a curve of a morphological return period with the same safety level (or return period) as for the hydrological data of bankfull discharge. This latter is a return period according to Table 4.2-1.

For the envelope method, data from surveyed cross-sections of the respective channel are required, where a protection structure is planned. Data can be made available from SWMC or from irregular bathymetric surveys of the ferry routes. Additional data have to be obtained from monitoring at the site of the planned structure prior to the planning phase. Only representative cross-sections, which are not affected by any river training structure, should be used.

To represent the morphological situation of the river bed during flood conditions for the design, the cross-sections should have been monitored at least at bankfull flow conditions (see Table 4.2-1). This corresponds also to the requirement that the monitoring should cover the full profile from thalweg to bank top. Otherwise the surveying with boats along the bankline cannot be carried out.

The envelope curve method is characterised by a collection of cross-sectional data originating from river reaches that all have either positive or negative curvature. All cross-sections should contain a clearly defined outer bankline. This is mostly given by a steep slope of the bank reaching the flood plain level. The collected data is then processed to refer the elevation of the measured bathymetric points to a uniform flood plain level. Additionally, the cross-sections are overlaid to have one point in common which is the outer bankline at flood plain level. The result is a collection of cross-sectional profiles as indicated in Fig. 4.3-1.

The envelope curve is derived by linking the bathymetric points with the deepest bed level within this dataset of processed cross-sections (thick line in Fig. 4.3-1). Finally, not a single cross-section may contain a point that is deeper than the envelope curve at this co-ordinate in the direction of the channel width. The envelope curve is used as the design cross-sectional profile. It represents an extreme near-bank profile of the channel that develops due to morphological processes at flood conditions. Moreover, the envelope curve method yields the water depth h'_0 in the deepest point (thalweg) below Flood Plain Level (FPL).

It is important to notice that the method must not be applied for the inner bank of a channel bend, as the total cross-sectional area would be overestimated and consequently the cross-sectional averaged flow velocities underestimated.

However, it is often not possible to survey cross-sectional profiles during hydraulic conditions that represent the morphological design conditions. Only from an extensive monitoring data base, the influence of extreme planforms on the design parameters can be taken into account. Other approaches for the determination of the required maximum water depth of the design cross-section are therefore described in Section 4.4.

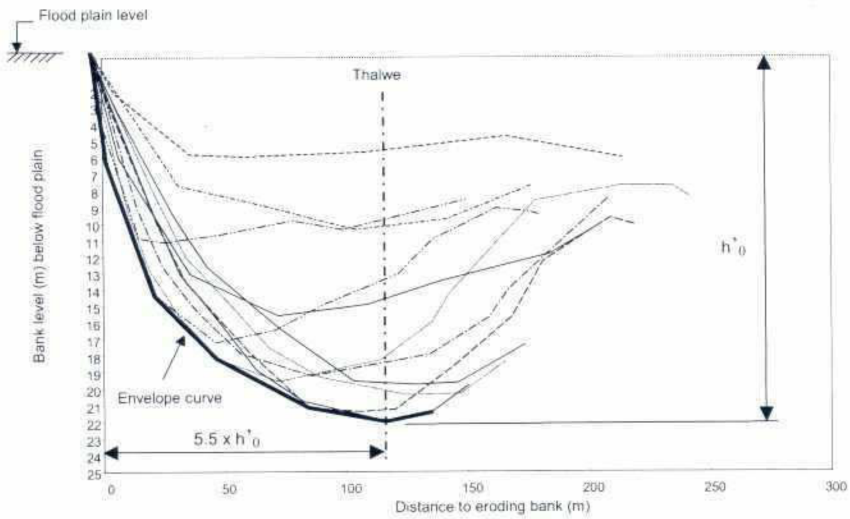

Fig. 4.3-1: Envelope curve of measured cross-sections of a channel bend

4.4 DESIGN CROSS-SECTION FROM REGIME EQUATIONS

4.4.1 Straight River Sections

The design cross-sectional profile can be achieved from an envelope curve only if the hydraulic conditions are representative and the number of surveyed cross-sections is sufficient. Alternatively, the size of the channel can be estimated by using a characteristic width and depth according to empirical regime formulas (see for example Special Report No.7 of the River Survey Project FAP 24, Delft Hydraulics and DHI, 1996 a). Therefore, the design channel is idealised by a rectangular cross-section as indicated in Fig. 4.4-1.

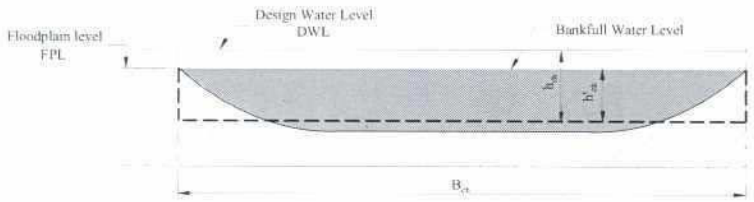

Fig. 4.4-1: Sketch of a cross-section in a straight river section

The cross-sectional channel area is defined as:

$$A_{ch} = B_{ch} \cdot h_{ch} \qquad \text{(Eq. 4.4-1)}$$

with

A_{ch} (m^2) cross-sectional channel area
B_{ch} (m) channel width between the banklines of bankfull discharge at FPL
h_{ch} (m) average water depth within the channel related to DWL

h'_{ch} is the average water depth within the channel related to FPL. The average channel width B_{ch} at FPL and the average bankfull water depth h'_{ch} are calculated from the bankfull channel discharge $Q_{b,ch}$ and the empirical coefficients c_1 to c_4 by regime relations:

$$h'_{ch} = c_1 \cdot Q_{b,ch}^{\ c_2} \qquad \text{(Eq. 4.4-2)}$$

$$B_{ch} = c_3 \cdot Q_{b,ch}^{\ c_4} \qquad \text{(Eq. 4.4-3)}$$

The empirical coefficients are listed in Table 4.4-1. The regime relations stem from Special Report No.7 of the River Survey Project FAP 24 (Delft Hydraulics and DHI, 1996 c).

Name		Jamuna	Ganges	Padma
Regime	c_1	0.40	0.28	0.28
$h_{ch} = c_1 Q_{b,ch}^{c2}$	c_2	0.26	0.29	0.30
Regime	c_3	8.97	9.97	4.76
$B_{ch} = c_3 Q_{b,ch}^{c4}$	c_4	0.57	0.555	0.62

Table 4.4-1: Regime relations for individual channels at bankfull flow conditions

The average water depth h_{ch} below DWL is calculated from the bankfull water depth h'_{ch} and the elevation of DWL and FPL as follows:

$$h_{ch} = h'_{ch} + (DWL - FPL) \qquad \text{(Eq. 4.4-4)}$$

Finally, the elevation of the river bed related to PWD can be calculated by subtracting the water depth h_{ch} from the elevation of DWL.

4.4.2 Bend Scour in River Bends

The approach used for straight river sections is rather simplified and does not sufficiently account for river bends, where the flow creates an asymmetrical profile with a bend scour at the outer side and a deposit at the inner side of the bend. The inner profile of the cross-section is indicated by a slope of the river bed gradually decreasing towards the inner bankline. The bend scour depth is mainly dependent on the channel width and the bend curvature.

(a) <u>Decisive Channel Width</u>

In case of river bends, the predefined width Bch of the channel does not cover the total width of the river bed between the locations where the Floodplain level (FPL) is reached. Instead, the inner bankline is defined by chars, which are dry at standard low water (SLW) but are inundated at bankfull water level. Consequently areas with shallow water at the inner bend are neglected (see Fig. 4.4-2).

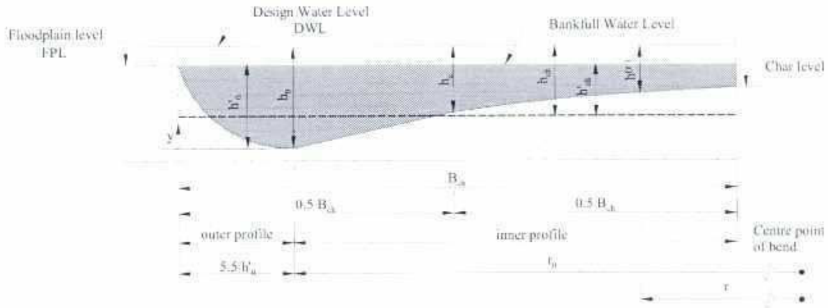

Fig. 4.4-2: Sketch of a cross-section of an axis-symmetric bend

(b) Bend Curvature

The bend curvature of the channel can be described by the radius of the channel centre line. The centre line in the cross-section is located at half the width B_{ch}. It is recommended to use a design bend radius of 2.5 times the channel width B_{ch}:

$$r_c = 2.5 \cdot B_{ch} \tag{Eq. 4.4-5}$$

Hickin and Nanson (1984) studied the bends in meandering rivers and found that under these circumstances the largest erosion occurs. The estimate has been preliminarily verified for the Jamuna river as part of a planform study.

However, if the subsoil of the river bed is not homogeneous but contains areas with cohesive layers, the estimate should be used with great care. Preference should be given to planform analysis then, as geo-technical effects can yield an extreme curvature with a smaller radius. In these cases, a planform analysis of satellite images is an adequate procedure to define the design channel curvature.

(c) Bend Scour Depth

For the design of a planned protection structure, the bend scour depth $y_{s,bend}$, which represents the deepening of the riverbed along the thalweg, is a decisive parameter. To determine the bend scour depth, the following approach is recommended:

Thorne (1988) found an empirical formula for the total water depth h_0 in the deepest point of the thalweg in a river bend being a function of the radius in the centreline r_c and the channel width B_{ch}. Assuming that the water depth in the upstream straight river section equals the cross-sectional averaged water depth h_{ch} , the formula reads:

$$\frac{h_0}{h_{ch}} = 2.07 - 0.19 \cdot \log\left(\frac{r_c}{B_{ch}} - 1.5\right) \tag{Eq. 4.4-6}$$

in which

h_0 (m) total water depth at thalweg below DLW
r_c (m) radius of bend centerline

The formula implies that $r_c > 1.5\ B_{ch}$. If the estimate $r_c = 2.5\ B_{ch}$ is used (see Eq. 4.4-5), Eq. 4.4-6 comes down to:

$$h_0 = 2.07 \cdot h_{ch} \tag{Eq. 4.4-7}$$

The scour depth y_s is defined as the difference between the water depth in the deepest point of the scour hole and the averaged water depth in the approach channel just upstream from the scour hole. Thus, the bend scour depth $y_{s,bend}$ can be written as:

$$y_{s,bend} = h_0 - h_{ch} \qquad \text{(Eq. 4.4-8)}$$

With the convention made before (see Eq. 4.4-4), h'_0 is understood as the largest water depth in the thalweg at bankfull discharge. From observations of cross-sections in the Jamuna river, an enveloping curve with the average slope of the outer bend profile between the deepest point in the thalweg and the outer bank at FPL was derived that holds the condition of 1:n with n = 5.5 (see Fig. 4.4-2). This is valid for sandy and cohesiveness soils (as prevailing in the Brahmaputra-Jamuna and other rivers of Bangladesh).

4.4.3 Outer Bend Profile

For the outer bend profile, an empirical formula has been derived based on enveloping curves from measured cross sections in the Jamuna river. The formula is deduced from the observation of bankfull discharges and is considered valid for other rivers in Bangladesh too. It reads:

$$h'(r) = h'_0 - h'_0 \cdot \left(1 - \frac{r_c + 0.5B_{ch} - r}{n \cdot h'_0}\right)^2 \qquad \text{(Eq. 4.4-9)}$$

with

h'(r)(m)		local water depth below FPL
h'_0	(m)	water depth at thalweg below FPL $h'_0 = h_0 - (DWL - FPL)$
B_{ch}	(m)	channel width
r_c	(m)	radius of bend curvature at channel centre-line
r	(m)	co-ordinate for radius of bend, here: $r_0 \le r \le r_c + 0.5B_{ch}$
n	(-)	cotangent for the natural slope of outer bend

As a first approach, for sandy, non-cohesive soils (as prevailing in the Brahmaputra-Jamuna and other rivers of Bangladesh), n = 5.5 can be chosen, which was deduced from the envelope curve analysis (see Fig. 4.3-1).

4.4.4 Inner Bend Profile

The evaluation of the inner bend profile is of rather small importance for the actual design of standardized protection structures. It may become essential when more detailed information on the local water depth and/ or the complete flow cross-section of a channel is required (e.g. numerical modelling and other methodical research studies).

4.4.5 Other Types of Scour

The river bed is scoured below its average level due to different processes. According to the investigations in the River Survey Project (Delft Hydraulics and DHI, 1996 b) the following types of scour can be distinguished apart from the above mentioned bend scour. The total scour occurring in nature is often a combination of these types, strongly depending on the local geometry of the river:

2+

(a) Confluence Scour

Confluence scour occurs where two channels meet. For the Jamuna River it is recommended to calculate confluence scour depth with the approach by Klaassen and Vermeer (1988). The authors evaluated historical field data and a survey from 1987 and found from statistical regression the following empirical relation for the confluence scour depth $y_{s,confluence}$:

$$y_{s,confluence} = h_1'' \left(0.292 + 0.037 \cdot \varphi\right) \qquad \text{(Eq. 4.4-10)}$$

with:

- φ (°) angle of incidence of anabranches or confluence
- h_1'' (m) upstream water depth below SLW from the average of both approach channels

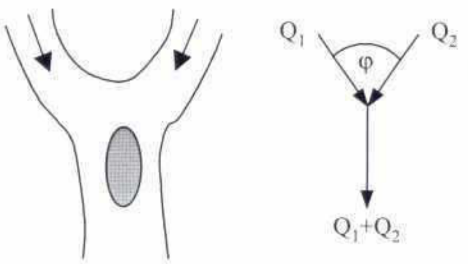

Fig. 4.4-3: Sketch of channel confluence with scour

The geometry of confluences changes easily, so that the design value of the incidence angle φ should not be measured from a temporary local situation. It is recommended to use a representative value of 45° for channels wider than 1 km and a representative value of 70° for channels narrower than 1 km.

The ratio of the discharges in the two anabranches should be estimated. For $1 < Q_1/Q_2 < 5/3$ the validity of the formula is proved (Q_1 is the higher discharge of the anabranches), whereas for higher ratios no clear dependency could be detected by the authors. Thus, in these cases the formula has to be used with great care.

Remark:
The water depth h_1'' upstream from the scour hole is not necessarily the cross-sectional averaged water depth in the upstream cross-section. In most cases it is represented by the water depth h_0 in the thalweg. This strongly depends on the width of the bend scour related to the size of the confluence scour hole. As the water depth has to be related to the water level SLW, the water depth h_0'' in the thalweg related to SLW has to be used.

(b) Protrusion Scour

Protrusion scour is caused by the flow acceleration along a structure. It occurs when the flow impinges on a structure or a relatively resistant bank. The flow is then concentrated within a smaller width, increasing its velocity. It is recommended to use the following formula, originally derived from the Mississippi river by Delft Hydraulics and DHI (1996 c,d):

$$\frac{y_{s,protrusion}}{h_1} = \frac{4 \cdot u_1^{0.33}}{(g \cdot h_1)^{0.165}} \qquad \text{(Eq. 4.4-11)}$$

in which

g	(m/s^2)	acceleration due to gravity
u_1	(m/s)	upstream depth-averaged flow velocity
h_1	(m)	water depth upstream from scour hole

The actual protrusion scour depends strongly on the local geometry and flow patterns. However, protrusion scour is not the decisive phenomenon for the design of a protection structure. The downstream local scour depth induced by the structure is of much higher importance. The protrusion scour can therefore be neglected for preliminary design purposes.

(c) Local Scour

Local scour is caused by the increased turbulence and the vortices in the decelerating flow downstream from a structure.

The depth of the structure induced local scour in a concave bend of a sandy cohesionless river bed (as applicable for the major rivers in Bangladesh) can be determined by the formula of Ahmad (1953):

$$h_1 + y_{s,local} = K \left(h_1 \cdot u_1 \cdot \frac{B_{ch}}{B_{ch} - b} \right)^{\frac{2}{3}} \qquad \text{(Eq. 4.4-12)}$$

in which

K	($m^{-1/3}s^{2/3}$)	empirical coefficient (Sub-Section 4.4.6)
B_{ch}	(m)	channel width upstream of the groyne
b	(m)	protrusion length (see Fig.4.4-6)
u_1	(m/s)	upstream depth-averaged flow velocity
h_1	(m)	water depth upstream from scour hole
$y_{s,local}$	(m)	maximum local scour depth

The empirical coefficient K has to be individually chosen according to the type of protection structure. Values for groynes and revetments are given in Section 4.4.6 (a) and (b) respectively.

The ratio $B_{ch}/(B_{ch}-b)$ considers the constriction due to the structure and represents a factor for the increase of the upstream flow velocity u_1. The protrusion length b equals to the groyne length reaching into the river (measured from the same bankline as B_{ch}). It has also to consider the permeability of the respective groyne section. However, if this length cannot be established, Fig. 4.4-4 can be used to estimate the factor $B_{ch}/(B_{ch}-b)$ in dependence of the water depth h_{ch} and the channel width B_{ch}.

With this, the depth-averaged flow velocity along the structure becomes $u = u_1 B_{ch}/(B_{ch}-b)$. According to observations from Breusers and Raudkivi (1991), the equilibrium scour depth depends only on the geometry of the structure if the depth-averaged flow velocity u is larger than $2 u_{cr}$. In this case, the grain size has obviously no direct effect on the maximum local scour depth $y_{s,local}$. For velocities below u_{cr}, the scour depth is also a function of the river bed grain size, so that the above formula can not be applied. Therefore, the following limits have to be considered:

$$u_{cr} \le u_1 \cdot \frac{B_{ch}}{B_{ch}-b} \le 2 \cdot u_{cr} \qquad \text{(Eq. 4.4-13)}$$

As for the confluence scour, the water depth h_1 upstream from the scour hole is not necessarily the cross-sectional averaged water depth h_{ch} in the upstream cross-section. In river bends it is represented by the water depth h_0 in the thalweg. However, this strongly depends on the width of the bend scour related to the size of the local scour hole.

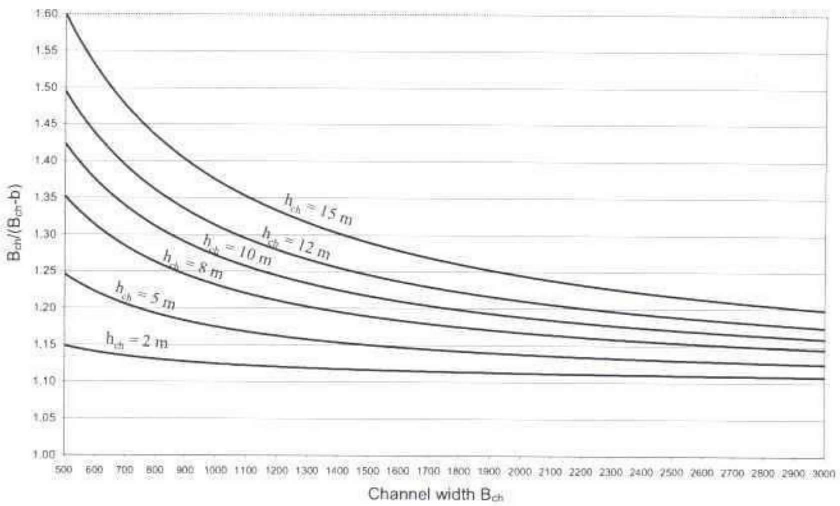

Fig. 4.4-4: Estimated constriction factor for local scour calculation

(d) Extra Structure Induced Scour

Some of the above mentioned scour types are not induced by the channel geometry but by bank protection or river training works. These structures along migrating river channels also produce morphological changes in a wider area as they form a local bank stabilisation. Channels become narrower, bends may become sharper and confluences of two or more channels may be formed close to the structure. Thus, these structure-induced morphological changes enhance the scour in several ways:

- The input of bank erosion material reduces the formation of near-bank scour. Stopping the input of bank erosion products by a structure will consequently eliminate this reduction and hence lead to deeper scour;
- Migrating rivers are overwide when point-bar deposition cannot keep up with bank erosion. The stopping of migration causes narrowing and associated deepening of the approach channels;
- The hindered migration deforms the bend. This increases the curvature and hence the bend scour of the channel along the predicted bend;
- The formation of a confluence, as a result of flow attraction by scour holes, produces confluence scour.

The scour enhancement due to these structure-induced morphological changes is still largely unknown. Most quantitative methods for the prediction of structure-induced scour hold for relatively stable rivers where the bank lines are fixed and hence the geometry does not change. In unstable rivers where bed levels and bank lines change rapidly, however, bank protection structures may cause these effects which are referred to as extra structure induced scour in the following. The extra scour occurs typically over the whole length of the protection structure or over the whole length of a river bend. This means that the area affected by extra scour is much larger than the area of a local scour.

Currently there are no quantitative methods to predict the extra structure-induced scour in rapidly changing rivers. For the design, it is therefore recommended to increase the total scour by 20% to account for the extra scour.

(e) Total Scour

Total scour results from the combination of the different scour types. It is recommended to calculate the total scour depth from a combination of bend scour, confluence scour, local scour and extra structure-induced scour.

The bend scour together with the extra scour acts in a much larger area (with a length of about the length of the protection structure) than the local scour. Therefore they act independently of each other and they can be added to each other (summation principle). However, generally confluence scour and local scour have very similar horizontal geometric dimensions in width and length, therefore, a strong interaction seems likely and the summation principle might result easily in an overestimation of the total scour depth.

202

It is recommended that the total scour depth due to interaction should be taken either from the summation of bend scour and local scour or from the combination of bend scour and confluence scour. The maximum value of both should be selected and added to the bend scour. To account for extra scour, the sum is multiplied with a factor of 1.2. This is expressed in the following formulae for the maximum design scour depth. Below the below the cross-sectional averaged bed level with the water depth h_{ch} the total scour is $y_{s,ch}$ (see Eq. 4.4-14a). Related to the deepest point in the thalweg with the water depth h0, where the bend scour is already included, the total scour is $y_{s,0}$ (see Eq. 4.4-14b):

$$y_{s,ch} = 1.2 \cdot \left(y_{s,bend} + \max \left[y_{s,local} \; ; \; y_{s,confluence} \right] \right) \qquad \text{(Eq. 4.4-14a)}$$

$$y_{s,0} = 0.2 \cdot y_{s,bend} + 1.2 \cdot \max \left[y_{s,local} \; ; \; y_{s,confluence} \right] \qquad \text{(Eq. 4.4-14b)}$$

with $y_{s,bend} = h_0 - h_{ch}$ according to Eq. 4.4-8.

4.4.6 Local Scouring at Protection Structures

(a) Local Scouring at Groynes

Near permeable groynes the following scour phenomena can be distinguished:

1. Local scour hole downstream from the tip of the groyne;
2. Local scour around the piles (groyne without a bed protection);
3. Local scour downstream from the transition from a permeable to an impermeable section;
4. Local scour upstream from the groyne (protrusion scour).

Ad 1: The local scour hole downstream from the tip of the groyne has received most attention, since it is most important for the design of the structure. It will be treated in detail to derive y_s.

Ad 2: The small local scour hole around piles is well known in the literature and the maximum depth is about 2 to 3 times the pile diameter if the water depth is larger then the pile diameter and if the groyne was designed without a bed protection (Breusers and Raudkivi, 1991).

Ad 3: In an analogous manner, an abrupt transition from an impermeable part to a permeable part can cause the development of a secondary scour hole close to the bank (Fig. 4.4.-5). Therefore a gradual transition is recommended to prevent the development of such a secondary scour hole.

Ad 4: This type of scour is not included in the design procedure of the structures, because of its small size.

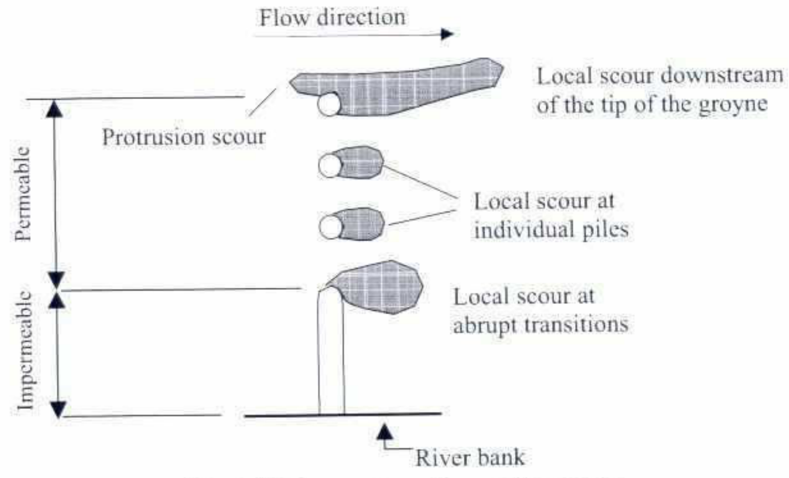

Fig. 4.4-5: Scour pattern of permeable groynes

As described before, bend scour can interact with the local scour at the tip. In theory, also an interaction scour between confluence scour and local scour, or confluence scour, bend scour and local scour is possible, and might result in very deep scour. Therefore the design procedure uses the total design scour depth, explained in Section 4.4.5 (Eq. 4.4-14).

The total scour depth y_s, the local structure-induced scour depth $y_{s,local}$ behind the tip of the groyne can be calculated by Eq. 4.4-15:

$$h_1 + y_{s,local} = K\left(h_1 \cdot u_1 \cdot \frac{B_{ch}}{B_{ch} - b}\right)^{\frac{2}{3}} \qquad \text{(Eq. 4.4-15)}$$

For explanations to the symbols and limits of the formula see Section 4.4.5.

For permeable groynes, the coefficient K is a combination of the empirical coefficient $K_{structure}$ $[s^{0.67}/m^{0.33}]$ depending on the permeability of the groyne and two dimensionless factors to account for bed protection and floating debris:

$$K_{total} = K_{structure} \cdot K_{bed\ protection} \cdot K_{floating\ debris} \qquad \text{(Eq. 4.4-16)}$$

Values for the single factors can be taken from Table 4.4-2.

Permeability at the tip (%)	$K_{structure}$ ($s^{0.67}/m^{0.33}$)
0	2.4
50	1.6 to 2.0
60	1.5 to 1.8
70	1.4 to 1.6
80	1.2 to 1.3
	$K_{bed\ proteciton}$ (-)
without bed protection	1.0
with bed protection around piles	1.1
	$K_{floating\ debris}$ (-)
without floating debris	1.0
floating debris ≤ 1 m thickness	1.2
floating debris > 1 m thickness	1.3

Table 4.4-2: Empirical coefficients K for groynes

(b) Local Scouring at Revetments

The local scouring in front of and downstream from a revetment (see Fig. 4.4-5) can be treated analogous to the local scour at the abrupt transition at the tip of an impermeable structure (see Table 4.4-2) in combination with bed protection. It is therefore recommended to use Ahmad's formula (Eq. 4.4-12) with $K_{structure}$ = 2.6. Floating debris does not have to be considered with an additional factor.

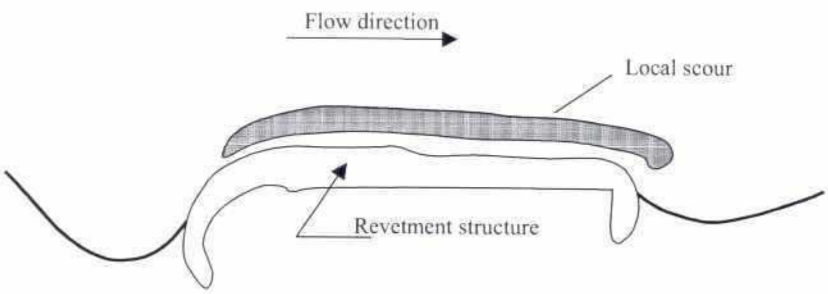

Fig. 4.4-6: Scour pattern of revetments

If the protrusion length b cannot be derived differently, the constriction factor $B_{ch}/(B_{ch}-b)$ can be obtained from Fig. 4.4-4. As the revetment does not block the outer bend, the constriction is not fully developed. To account for this effect, a channel width of B_{ch} = 3000 m is to be applied in the given diagram. This yields a constriction factor between 1.1 and 1.2. However, the approach should be used with great care for changes in approach conditions due to the response of the river to the structure.

4.4.7 Scour Velocity

The time-dependent development of the scour depth is referred to as scour velocity. It determines the risk of the loss of stability of the structure by flow slides, as fast scouring can trigger slides. Moreover, it has to be evaluated, as slow scouring may have the effect that the values predicted by Ahmad's formula (Eq. 4.4-12) would never be reached as the flow conditions change before the equilibrium state can develop.

The formula for the characteristic time t_1 (in hours) at which the scour depth equals the upstream water depth ($y_s = h_1$) reads

$$t_1 = \frac{k_t \cdot h_1^{\ 2} \cdot \Delta^{1.7}}{(\alpha_t \cdot u_1 - u_{cr})^{4.3}} \qquad \text{(Eq. 4.4-17)}$$

in which

k_t	(-)	empirical constant = 330
h_1	(m)	upstream water depth
Δ	(-)	relative density of sediment, usually $\Delta = 1.65$
α_t	(-)	coefficient depending on the geometry upstream of the scour hole
u_1	(m/s)	upstream depth-averaged flow velocity
u_{cr}	(m/s)	critical flow velocity for initiation of motion = 0.4 to 1.0 m/s (see Section 4.4.5)

The coefficient α_t was investigated by Breusers and Raudkivi (1991) in physical model tests on scours behind vertical boards. It varied for the three-dimensional scour case between 1.6 and 1.9 for rough and smooth bed protection upstream of the scour respectively.

If the expected maximum scour hole depth $y_{s,design}(t)$ is smaller than the upstream water depth h_1, then the following simple relationship can be applied (Hoffmans and Verheij, 1997):

$$\frac{y(t)}{h_1} = \left(\frac{t}{t_1}\right)^{n_t} \qquad \text{(Eq. 4.4-18)}$$

in which

h_1	(m)	water depth upstream from scour hole
$y(t)$	(m)	time-dependent local scour depth
t	(h)	time
n_t	(-)	empirical coefficient

For three dimensional scour holes, the empirical exponent n_t is 0.6 to 0.8 (Breusers and Raudkivi, 1991). The average scour velocity can be estimated for any scouring duration of length t with $y(t)/t$. In the initial phase the average scour velocity should be smaller than 2 m/day, as faster scouring can trigger slides and produce sudden bank erosion.

207

4.5 DESIGN FLOW VELOCITIES

4.5.1 Introduction

The design flow velocities in the approach flow of a planned protection structure can be determined in various ways. It should not be estimated only for the deepest point of the approach channel cross-section but along the whole bank through:

- Statistical analysis of observed flow velocities;
- Simulation in a 2-D (depth-averaged) mathematical model or in a physical model;
- Theoretical calculation methods.

The steeper water level gradients during the rise of the flood imply that the maximum flow velocities occur a few days before the maximum water level is reached. Therefore, it is physically more accurate, to consider a peak flow instead of the design discharge for the evaluation of the design flow velocities. Thus, the peak flow theoretically occurs a few days before the design discharge passes the site.

From experience, the maximum flow velocities occur 3 to 4 days before the maximum water level is reached, thus during the rising limb of the flood. It can be shown that the rise of the water level increases the water level gradient. As the river bed becomes smoother, this effect results in an increase of the flow velocities. However, it has to be noted that the flow velocity can also increase to its maximum as a steep water level gradient develops during a cut-off. The days, when flow velocities should be observed for statistical analysis, are consequently during the rising limb of the flood.

If the statistical analysis of observed flow velocities cannot be carried out, preference should be given, with respect to the simplicity of the approach, to the theoretical calculation methods presented in the following subsections.

4.5.2 Cross-sectional Averaged Flow Velocity

With the geometric parameters of the approach channel, i.e. the cross-sectional averaged water depth h_{ch} and the bankfull channel width B_{ch} (see Fig. 4.4-1), the flow velocity averaged over the channel cross-section can be derived from the design channel discharge Q_{ch}. The increased flow velocities, which occur during peak flow, are treated by assuming the total design channel discharge passing through the area between the riverbanks. Consequently, the flow velocity on the flood plains is neglected:

$$u_{ch} = \frac{Q_{ch}}{B_{ch} \cdot h_{ch}} \qquad \text{(Eq. 4.5-1)}$$

4.5.3 Local Depth-averaged Flow Velocity

The flow pattern in a meander bend is strongly three-dimensional and a protection structure at its outer bend makes this flow pattern even more complicated. Thus, for the flow velocities along or behind a structure, individual approaches have to be used. For the design of the protection structure, only the depth-averaged flow velocities in the thalweg and at the outer bend have to be estimated. Additionally, the velocity reducing effects of permeable groynes have to be calculated for the design of a groyne field.

For a straight river reach, the flow velocity in the deepest point can be calculated with the same approach as for the thalweg of a river bend (see Eq. 4.5-2). The distribution of the flow velocity in transversal direction has to be analysed further as for an outer bend (see Eq. 4.5-3). However the formulas are described for the case of a meander bend, as this is the dominant type of river reach.

(a) Flow Velocity in the Thalweg of a River Bend

The depth-averaged flow velocity in the thalweg is estimated by the Chézy formula from the given cross-sectionally averaged flow velocity u_{ch}. The ratio of both is given by:

$$\frac{u_0}{u_{ch}} = \sqrt{\frac{h_0}{h_{ch}}} \qquad \text{(Eq. 4.5-2)}$$

where

h_0	(m)	water depth in the thalweg
h_{ch}	(m)	cross-sectional averaged water depth below DWL
u_0	(m/s)	depth-averaged flow velocity in the thalweg
u_{ch}	(m/s)	cross-sectional averaged flow velocity

(b) Flow Velocities at the Outer Bend

The approach flow velocity above the outer bank slope of a river bend (outer profile in Fig. 4.4-2) is estimated by the Chézy formula and the Colebrook-White formula for steady uniform flow. So the variation of the local roughness of the underwater bank slope is considered. The ratio of the local depth-averaged flow velocity u(r) above the underwater slope to the calculated depth-averaged flow velocity u_0 in the thalweg is given by:

$$\frac{u(r)}{u_0} = \frac{\log\frac{12h(r)}{k_s(r)}}{\log\frac{12h_0}{k_{s,0}}}\sqrt{\frac{h(r)}{h_0}} \qquad \text{(Eq. 4.5-3)}$$

where

h_0	(m)	water depth in the thalweg
$h(r)$	(m)	local water depth above the underwater bank slope
$k_{s,0}$	(m)	Nikuradse equivalent sand roughness in the thalweg
$k_s(r)$	(m)	local Nikuradse equivalent sand roughness of underwater bank slope
u_0	(m/s)	depth-averaged flow velocity in the thalweg
$u(r)$	(m/s)	local depth-averaged flow velocity above the underwater bank slope

207

For the Nikuradse equivalent sand roughness the grain size D_{90} can be used if no bedforms are present. In the realistic case that ripples and dunes affect the hydraulic roughness of the river bed, a Nikuradse equivalent sand roughness of $k_s = 20$ mm is to be chosen. If the u_0 is known from Eq. 4.5-2 then u(r) can be calculated from Eq. 4.5-3. The latter describes the flow velocities approaching the protection structure. If the calculated velocities along the bank are above the critical flow velocity for bank erosion (see Section 4.5.5), then it is likely that an unprotected bank will subsequently erode.

(c) Reduction of Flow Velocities by Groynes

The main effect of a permeable groyne is the reduction of flow velocities in the flow passing it. The here presented method to estimate this reduction is based on the assumption that pile rows are used for construction.

In the following, a control volume bordered by the flow lines that intersect the permeable groyne in the centre of the spaces between a pile and its neighbouring piles is considered. Subscript 1 refers to the upstream boundary of the control volume and subscript 2 to the downstream boundary. It is assumed that the riverbed does not vary in the direction of the flow ($A_1 = A_2$) and that the flow velocity at the upstream boundary is equal to the approach flow velocity u_0 above the toe of an outer bend ($u_{ch,1} = u_0$).

From the momentum equation the following can be derived:

$$\frac{u_{ch,2}}{u_0} = \sqrt{1 - \frac{1}{2} \cdot C_D \cdot \frac{A_p}{A_1 \cdot \cos\gamma}} \qquad \text{(Eq. 4.5-4)}$$

in which

- $u_{ch,2}$ (m/s) downstream cross-sectional averaged flow velocity
- u_0 (m/s) depth-averaged flow velocity in the thalweg
- C_D (-) drag coefficient
- A_p (m^2) cross-sectional area of piles
- A_1 (m^2) upstream cross-sectional area
- γ (°) direction of the separation flow line including effect of oblique flow attack

Remarks:

- The subscript p in above formula refers to the pile. So the flow blockage by the pile row is A_p/A_1.
- If floating debris is present, A_p should be replaced by $A_p + A_{fl}$ where A_{fl} is the cross-sectional area blocked by floating debris.
- γ is the angle between the flow lines and a line perpendicular to the groyne axis. This angle is determined by the expansion angle and the oblique approach flow.

The reduction of the flow velocity is obviously a function of the drag coefficient C_D. It is recommended to use the following:

- $C_D = 0.7$ for smooth single piles in strong flows (design flow velocities 3 to 3.5 m/s);
- $C_D = 2.0$ or rough, closely spaced piles in moderate flows
  (approach flow velocities 0.5 to 0.9 m/s during monitoring).

When a series of permeable groynes is attacked by parallel flow, the flow velocity is reduced from groyne to groyne in downstream direction by repeated application of Eq. 4.5-4. However, lateral momentum transfer, expansion angle and oblique flow attack will affect this reduction of flow velocities adversely. Therefore, Eq. 4.5-4 should not be applied more than three times to prevent too optimistic estimates of the flow velocity reduction. That is why permeable groynes should be applied in series of at least three groynes to obtain a considerable reduction in flow velocities along the bank and to stop or to prevent bank erosion.

4.5.4 Vertical Flow Velocity Distribution

The above described flow velocities u, consider the average flow velocity along a vertical line. However, turbulence and shear flow effects yield a velocity gradient in vertical direction. A logarithmic velocity profile (Fig. 4.5-1) according to Eq. 4.5-5 can be assumed:

$$u(y) = u \cdot \frac{\ln\left(30\,\frac{y}{k_s}\right)}{\ln\left(11\,\frac{h}{k_s}\right)} \qquad \text{(Eq. 4.5-5)}$$

The velocity profile contains the depth-averaged flow velocity u in a distance of 0.63 times the total water depth h below the water surface, which is at $y = 0.37$ h. Using a Nikuradse equivalent sand roughness of $k_s = 20$ mm, the flow velocity u_s at the water surface can be estimated to $u_s = 1.16$ u. The velocity u_b at the river bed can be estimated with $u_b = 0.60$ u.

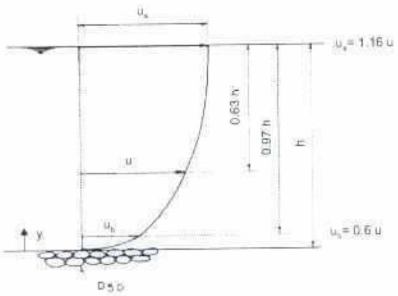

Fig. 4.5-1: Logarithmic flow velocity profile

The effects of wind and turbulence generated by interaction of the channel flow with the flood plains on the flow velocity profile can be neglected. Consequently the maximum flow velocity can be assumed to be located at the water surface as indicated in Fig. 4.5-1.

220

4.5.5 Critical Flow Velocity for Initiation of Motion

The critical flow velocity u_{cr}, is a depth-averaged flow velocity that induces the initiation of sediment motion. The u_{cr} depends on the local water depth and the subsoil (D_{50}, cohesiveness).

Instead of a critical flow velocity a critical shear stress exerted on the river bed near the bank can be used as a parameter to indicate a stable or eroding bank. The critical value of the Shields parameter θ_{cr} based on shear stress varies between 0.03 and 0.1 depending on the mean grain size D_{50} of the riverbed and bank. Following the Shields approach, a critical depth average flow velocity u_{cr} between 0.22 and 0.4 m/s can be determined. However, this approach is non-conservative because it holds only for uniform flow conditions and non-cohesive sediment with uniform grain size.

Due to the constrictions (mainly the cohesiveness of the material), the critical depth-averaged velocity u_{cr} according to Shields has to be increased. From the experience, it is recommended to use 0.4 m/s for shallow water and 1.0 m/s for deep water. The velocity u_b above the river bed can be estimated with $u_b = 0.6$ u (see Fig. 4.5-1), with the depth-averaged velocity u. Consequently, it is assumed that the river bed motion is initiated if u_b is between 0.24 m/s and 0.6 m/s.

4.6 OTHER BOUNDARY CONDITIONS

4.6.1 Floating Debris

In Bangladeshi rivers floating debris usually originates from water hyacinths and banana tree trunks. In a groyne field, it can be assumed that floating debris piles up to a maximum depth (design depth) of about 2 m below any water level at the most upstream groynes. In general, the thickness of the layer of floating debris depends on the flow velocity in the channel.

The highest amount of floating debris can be observed during peak flows, because during a peak flow new flood plain areas upstream form the groynes are inundated and these areas are often covered by water hyacinths that start to float with the flow. This means that floating debris can be observed when the water level rises above the flood plain level.

Two major effects of floating debris are known:

1. By accumulation against the permeable groyne, floating debris exerts considerable forces on the constructional element, mostly piles. It is therefore recommended to avoid this effect by designing the top of the piles suitably below the design water level. This reduces the load on the piles, while this reduction of pile height has only a small effect on the flow velocities in the groyne field.
2. Additionally, the permeability of the pile row at the water surface is reduced. The consequence is a redistribution of the approach flow velocity. Blockage of the permeable groynes, will increase the flow velocities at the tip of the groyne, because more discharge will pass through the channel. The increased velocities cause deeper scour holes.

Both effects are treated in the design of permeable groynes (see Chapter 6). At the upstream termination of a revetment structure floating debris does not need to be considered since it is not expected to influence the flow velocities and subsequently the scour development along the structure.

4.6.2 Ship Impacts

Major parts of the Jamuna river are not yet utilised for intensive inland waterway transport. However, accidental impact loads to the most exposed groyne piles, such as ship impact (e.g. caused by oil barges plying on the river) should be considered. Furthermore, permeable groynes projecting into the river may present a danger, in particular during high flood water levels, for country boats, inland waterway vessels and bamboo/timber rafts navigating on the river. Dependent on the location, safety marks at the groyne head should be provided.

Revetments and bed protections around or near ferry ghats may be subjected to additional hydraulic loads caused by manoeuvring ferry boats. Ship propellers generate high jet velocities in the column water, called "screw race" that may impinge on the bed or bank of the waterway. It is normally only a significant load at moorings or a ferry ghat where serious scour on the bank face can result from a vessel starting from stationary or whilst manoeuvring near the bank. As an indication of bed velocities due to the action of the propeller, an additional flow velocity above the bed between 2.0 and 2.5 m/s is assumed. However, the level of damage is proportional to both the screw race and the duration over which the screw race is attacking the area. For taking appropriate consideration of this effect, the frequency of manoeuvring has to be observed.

4.6.3 Earthquake Loads

Seismic aspects were described in Chapter 1. The stability of the river banks and the protection structure is affected by possible horizontal seismic acceleration. Its magnitude has to be estimated according to the seismic zoning map of Bangladesh.

4.6.4 Surface Loads

Surface loads on the protection structures will occur particularly during the construction phase. If heavy trucks and other large machinery are used, maximum acceptable loads must be confirmed. For strategic stockpiling it is recommended to allow for a surface load on the earth dam areas (except the sloped part) corresponding to a 1.5 m high stockpile of boulders. On a revetment structure this load has to be assumed on the entire crest area.

4.6.5 Soil Conditions

Detailed subsoil investigations have to be carried out at the potential site location. The evaluation of field samplings and laboratory tests presents finally the soil mechanical parameters in different soil layers:

- Relative density based on SPT n-values;
- Soil classification;
- Soil properties:

γ (kN/m^3)	unit weight
γ' (kN/m^3)	submerged unit weight
φ (°)	angle of internal friction
c' (kN/m^2)	cohesion(kN/m^2)
k_s (m/s)	coefficient of permeability

- Grain size distribution:

d_{60} (m)	grain size with 60% sieve passage
d_{50} (m)	median grain size
d_{10} (m)	grain size with 10% sieve passage
U (-)	coefficient of uniformity $U = d_{60}/ d_{10}$

The above soil properties should be somewhat conservative to cover variations of the subsoil conditions.

From the experience from sites at the Jamuna river, the following tendencies are valid for the soil classification:

(i) Close to the flood plain surface slightly cohesive soil stratum prevails up to a depth of about 5 m on an average;

(ii) Below the upper layer loosely to medium dense deposited micaceous silty fine sand exists until a depth of about 20 m, followed by

(iii) Medium dense to dense silty fine sand, which can be assumed to prevail up to a depth of interest for bank erosion prevention measures.

Despite the fact that the soil conditions along the Jamuna river may be considered somewhat uniform, it must be a standard requirement for any structure to be built that for its final design the soil properties are being determined on the basis of detailed subsoil investigations. In addition the ground water table must be determined.

If data are not available, for SPS the conservative conditions presented in Table 4.6-1 may be used as a first approximation.

Parameter	Flood Plain Level to SLW	SLW to SLW – 5m	Below SLW – 5m
Soil classification	Clayey sandy silt to silty sand; (CL – ML)	Fine to medium sand, silty, partly clayey (SM)	Sand
Grain size distribution (mm) d_{60} d_{50} d_{10}	 0.03 to 0.09 0.02 to 0.07 0.003 to 0.03	 0.11 to 0.20 0.09 to 0.22 0.04 to 0.07	 0.20 to 0.26 0.15 to 0.22 0.06 to 0.08
Coefficient of uniformity $U = \frac{d_{60}}{d_{10}}$	3 to 10	3 to 4	3 to 4
Coefficient of permeability (m/s)	-	$3 \cdot 10^{-5}$	
Angle of internal friction φ' (°)	25 to 27.5	27.5 to 32.5	
Cohesion c' (kN/m²)	7 to 20		0
Unit weight/submerged unit weight γ/γ' (kN/m³)	18/8		

Table 4.6-1: Basic assumptions on soil properties (SPS)

5 DESIGN OF REVETMENTS

5.1 GENERAL ASPECTS

A revetment is a structural protection against wave and current induced loads covering the existing river bank or an embankment. A sound knowledge and clear understanding of the physical processes behind structural failures is required for a safe and economic design of revetments. To allow also for consideration of economic aspects when designing a revetment structure, it is essential to distinguish between different failure modes and taking into account the respective frequency/ duration of the design loads (Chapter 4).

A revetment normally comprises the different layers of a protection system, which is covering a natural slope (subsoil) or an artificial dam (core material). A typical multi-layer design of a revetment system is shown in Fig. 5.1-1.

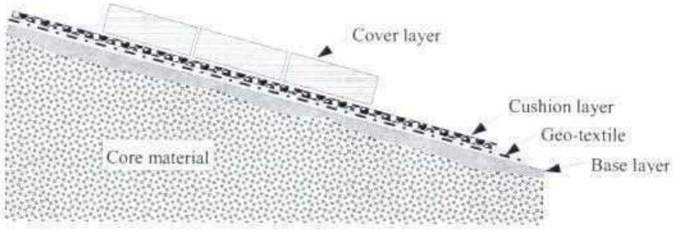

Fig. 5.1-1: Typical multi-layer revetment system

The cover layer must persist the assumed design impacts (mainly current and wave), whereas the intermediate layers between cover and core material are required for drainage and filtering to allow for a stable foundation of the overall system. Furthermore, in certain cases a cushion layer is required to protect the geo-textile and a base layer which is used to level the existing subsoil or core material.

However, a revetment structure does also include a toe protection and a crest layout. The toe of a revetment structure requires special attention during the design. If overtopping by waves or by flow is to be expected, the stability of the structure crest and the inner slope of the revetment also needs appropriate protection.

In general, the stability analysis of a revetment should comprise the following general design criteria:

- the surface of the individual elements of the cover layer should be sufficiently resistant against abrasion by wave and current attack (surface resistance criterion)
- the slope normal weight of the revetment cover layer has to be larger as compared to the uplift pressure caused by waves or resulting from excessive pore pressure (uplift criterion)

- the revetment should be designed that no slides under frequent hydraulic loads occur (sliding criterion)
- the revetment including filter layers and subsoil must be stable as a whole (equilibrium criterion)

These aspects are considered by the structural design of standardized protection structures (pre-defined slopes, toe protection, etc., Section 5.2). The stability criteria for individual protection units are covered by empirical or semi-empirical formulae, which are based on stability tests in laboratories and field, and allowing for a definition of the required unit weight or layer thickness (as discussed in Sections 5.4 to 5.6).

5.2 LAYOUT CONSIDERATIONS

5.2.1 Design Concept

The design concept of the recommended revetment structures is strongly motivated by construction constraints. In this regard, main focus was put on the construction of all revetment components on the dry flood plain during off season. It is assumed, that all individual elements of the toe protection remain stable under the prevailing design flow and wave conditions, but on the other hand, small redistribution of the material is of minor consequence. With increasing scour depth in front of the structure, the toe protection elements start to proceed down the scour hole, thus protecting the river slope, which is estimated to be about 1V:2H in case of the maximum equilibrium scour depth. The minimum number of protection elements to allow for a theoretical coverage, equivalent to a layer thickness of 1.5 D_n (nominal unit dimension), is dependent on the bank normal slope length. To reduce the vertical distance towards the maximum developing scour hole in front of the revetment structure and thus to limit the amount of material needed, the horizontal datum of the toe protection is chosen as low as possible (SLW +0.5m). For that reason a temporary cofferdam during construction is advantageous, to protect the uncompleted revetment structure against unexpected early floods.

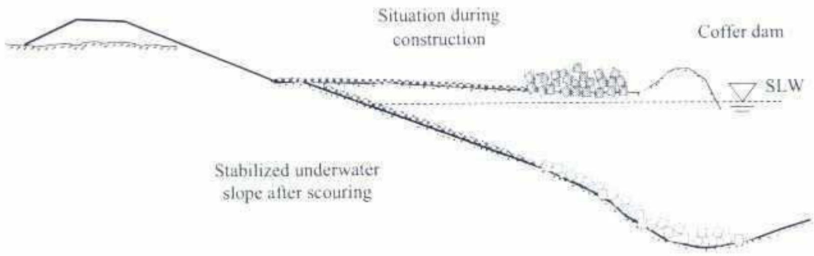

Fig. 5.2-1: Initial situation after construction and assumed profile development due to scouring in front of the revetment (sketch)

A basic parameter to be dimensioned is the characteristic unit size or layer thickness of the revetment armouring, dependent on the material to be used. Since the impacts from currents and wave loads are forced by specific physical mechanisms, two different approaches for the design against wave loading and against current induced shear stress are required. Dependent on the prevailing conditions, the larger values computed by the given formulae should be applied for the design.

Nevertheless, it should be kept in mind, that wave attack is considered significant only for the revetment cover layer and some parts of the launching apron (i.e., components above SLW). The falling apron is not susceptible to wave loads, because wave breaking at this position only occurs at lower water depth, which is limiting the expected wave height (restricted fetch and water depth). In addition, moderate movement/ dislocation of single armour units is tolerable. At the launching apron, wave loads might get important in case the elements of the falling apron have already proceeded towards lower parts of the bank slope, thus exposing the launching apron to larger water depth.

5.2.2 Length and Alignment

The length and alignment of a revetment structure unquestionably must be chosen in view of the area, which is to be protected against continuing bank line erosion. Nevertheless, some general policies should be regarded with respect to structure costs: (i) due to the comparatively expensive structure terminations, a certain minimum bank parallel structure length should be kept in the design (about 600 - 800m), whereas (ii), the maximum length should be defined by analysis of the costs and benefits of the projected structure, which is dependent on the socio-economic background of that particular location. Furthermore, the extend of the structure is restricted by the construction capacity within a dry season period and by the fact, that with increasing length of protected bank line, the possible negative impacts on the sediment balance at downstream locations might also get more evident.

In context with the interference between structure and flow, the structure alignment (defined as the angle between structure front and the course of the main river or of the approach channel) is also of importance. To reduce turbulences due to flow separation, it is in general recommended to arrange the structure at a slight angle (approx. 3 to 5 degrees) to a parallel line, both, relative to a major branch and to the main river course (the downstream end of the structure is pointing towards the channel). Therefore, the recommended alignment will always involve certain compromises regarding the optimum under different and often changing morphological situations.

In particular for rivers with a wide active flood plain (AFP) and strongly meandering branches (e.g., the Jamuna river) it can be stated, that a revetment structure should not be placed at location too far from the peripheries of the AFP, because an excessive upstream embayment, putting also the existing upstream flood embankment at risk, may be possibly followed by loss of the structure.

5.2.3 Terminations and Transitions

Despite a well-designed revetment, the construction is only as strong as the weakest section. Therefore, special care is required for the design of any transitions, terminations and joints. Special measures to limit the vulnerability of a revetment termination to erosion are crucial, because erosion damages are often initiated at discontinuities between protected and unprotected reaches of a river bank. An extension of the revetment beyond the point of active erosion is in general not feasible for budgetary reasons, especially at highly mobile meandering or braided rivers. Hence, smoothly curved and sloped terminations at both ends of the structure are recommended, starting from the straight part and pointing away from the river (tip about normal to the bankline, see **Fig. 5.2-2**). The slope and the radius of the upstream termination will also influence the development and extent of a potential upstream embayment, which should be restricted to prevent from excessive scouring due to the increased protrusion of the structure.

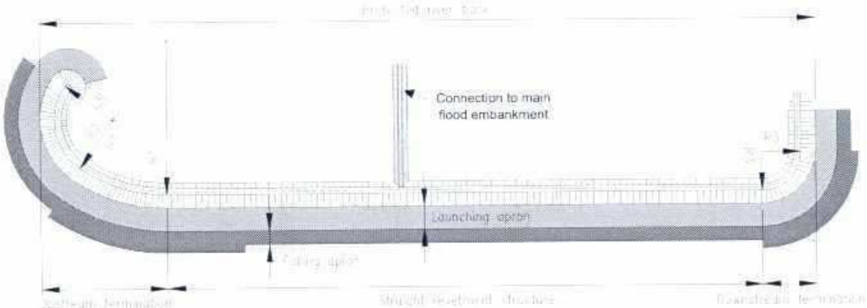

Fig. 5.2-2: Typical revetment structure (plan view)

For that reason the minimum radius R1 should not be chosen smaller than 250 m and, if possible, to be extended to 400 m, whereas for the downstream termination a radius R3 of 100 m to 150 m seems appropriate.

Though it seems advantageous to use only one type of cover layer from the toe to the crest of a revetment, different materials and execution principles are usually applied for the different components of the structure, such as toe protection and main protection in the area of heavy wave and current attack. Joints and transitions between different sections should be avoided as much as possible, in particular for different elastic and plastic behaviour and for differences in permeability of the materials used. In case transitions parallel to the bank (and to the stream lines) are inevitable, the discontinuities should be sufficiently protected by increasing the cover layer thickness or by grouting loose cover layer elements with bitumen/ cement to minimize the risk of damages.

227

5.3 CROSS-SECTION AND STRUCTURAL COMPONENTS

5.3.1 Cross-Section

The required height of the revetment structure crest is defined by the design high water level (DHW, see Chapter 4) plus a certain freeboard, which includes other aspects like wind set-up, wave run-up and further safety margins (Fig. 5.3-1). Due to rather small differences in design high water levels for return periods of 25 or 100 years (e.g., about 0.3 to 0.5m for certain locations at the Jamuna river), the respective increase in structure costs is limited, but has to be assessed during the planning process. The decision on the design water level should be risk based, for exceptionally important locations (SC4) a high water level referring to a 100 years return period is recommended.

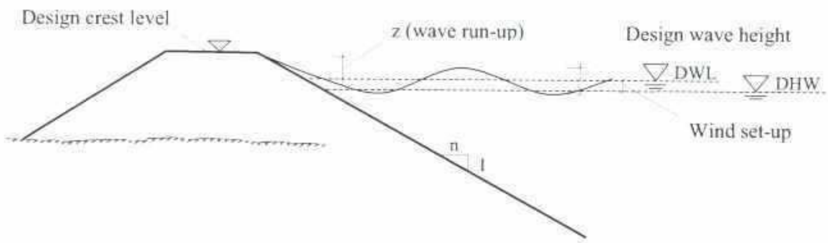

Fig. 5.3-1: Definition of revetment design crest level

Following Saville (1958, in Shore Protection Manual, 1984) the relative wave run-up z/H of comparatively short waves (about $H/gT^2 > 0.008$) at smooth 1 in 3 slopes can be estimated to $1.0 \leq z/H \leq 1.4$. Dependent on the wave height distribution higher run-up should be expected for the highest waves of the wave spectrum, whereas increased surface roughness and smaller slope angles would reduce the wave run-up. For structure categories SC2 and SC3 a relative wave run-up of $z/H = 1.2$ is recommended. A wind induced water level set-up is considered rather small in case of inland waterways but has to be investigated for estuaries.

The crest width has to be defined in terms of the overall structural stability (e.g. seepage) and possible multi-purpose use of the structure crest (road, stockpiling of extra material, etc.). For standardized revetment structures the crest width is given at a value of 4.5m, whereas the inner and outer slope has to be build in 1V : 3H to allow for an improved sliding resistance and overall stability of the structure.

The minimum distance of the revetment crest from the actual bankline at the beginning of construction is defined by the expected bank erosion during structure implementation, the width of launching and falling apron and of the coffer dam as well as by the horizontal datum of the toe protection, which is affecting the length of the revetment slope (see Fig. 5.3-2). Since the construction of all components is planned to be carried out in dry condition (approx. SLW + 0.5m) the horizontal datum of the toe protection is dependent on the topography of the projected location.

The core material (usually sand to silty/ clayey sand) is recommended to be taken directly or nearby from the structure site, if the required quality is available (refer to Chapter 3). Preferably, the material from excavation of the toe protection area should be used. The revetment crest should be protected by compacted khoa material to prevent from rain cuts and deterioration of the surface due to other impacts. For the inner slope (landward side) grass or reed might be sufficient if the plants are properly attended and wave action or wave overtopping is not being expected. A typical cross-section of a standardized revetment structure is given in Fig. 5.3-2.

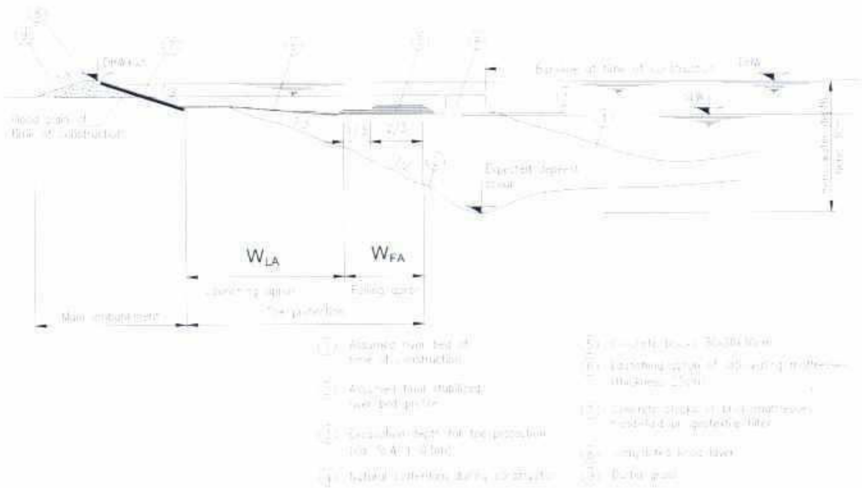

Fig. 5.3-2: Typical revetment structure (cross-section)

5.3.2 Cover Layer of Upper Revetment

The cover layer must provide protection against erosive forces of currents, wave action and other external effects. Most important properties of the surface layer, which affect the interaction between fluid (waves, currents) and structure, are

- Roughness: induces local friction and thus, is followed by reduction of flow velocities and wave run-up (surface of cover layer)
- Permeability: governs the water exchange and pressure balance between the core and the revetment surface (in context with filter layer)
- Flexibility: determines the ability of the revetment layers to accommodate minor deformations due to settlement, loss or migration of underlying material, and thus to maintain the composite integrity of the revetment (composite layers)

The stability of protection units against hydraulic and other loads is mainly defined by the respective mechanical strength, the (submerged) weight of the units as well as the friction/interlocking between individual elements and friction between the different layers of a revetment. Various elements and materials are used for cover layers including loose boulders and blocks as well as mattress systems, asphalt layers, etc. For cover layers of standardized revetments concrete cubes (CC-blocks) are recommended, which are resistant against various impacts and have the advantage of uncomplicated manufacturing, possible prefabrication and of being applicable for different structure components. The single layer CC-blocks should be placed in an orderly manner, to provide a uniform weight on the filter-layer and the subsoil.

Fig. 5.3-3: CC-blocks used as revetment cover layer

5.3.3 Filter Layer

The stability of the cover layer strongly depends on the type and composition of the filter and other intermediate layers (compare Fig. 5.1-1). Since the revetment layer system is acting as a whole, instability of the sub-layers and/or the subsoil due to erosion can lead to failure of the cover layer. The most important functions of a filter are

- to prevent from migration of subsoil particles out of the bank slope (filtering)
- to prevent from sinking of cover layer units into the core material (separation)
- to allow water exchange between core and surface in order to prevent from excessive pore pressure during wave run-down and in case of rapidly decreasing water levels

The filter layer may also act as a drainage zone parallel to the revetment slope and provides a preliminary protection of the structure core during construction. Furthermore, the filter material must be capable to withstand possible consolidation (settlement) of the subsoil after construction.

In general, either granular filters, made of loose, bounded or packed grains or fibre filters, made of synthetic materials are used. A granular filter is formed by layers of grain of different size according to the filter criterion, which relates the grading of the filter to that of the subsoil and of the cover material. It is necessary to check the filter uniformity to ensure that internal migration of fines does not occur. Granular filters must keep a certain layer and overall thickness to allow for a stable and efficient filtering.

Appropriate fibre filters provide a relatively permeable layer, in which particles of the soil can be retained without the risk of blocking or clogging the filter. Geo-textiles, containing the maximum number of pores per unit volume are most suitable for this purpose. In contrary to granular filters, geo-textiles are capable of bearing certain tension stresses, which can contribute to the overall stability of the structure and is a pre-requisite, when applied as a sub-layer for launching apron, where extreme deformations of the subsoil are expected. The use of geo-textiles allows for a considerable reduction of the construction time, but at present the material needs to be imported (foreign exchange required).

5.3.4 Toe Protection

Toe protection is required in case currents and / or waves scour and undermine the toe of a bank or an embankment, which are likely to result in sliding of the slope, thus endangering the overall stability and function of the revetment. A widely used type of toe protection is a blanket of loose material, a so-called falling apron, as CC-blocks, boulders, etc. (Fig. 5.3-4). The material is placed over an area of appropriate width and thickness in front of the embankment or on the riverbed. It is assumed, that scouring of the apron starts at the most river-sided edge and progresses towards the berm in front of the slope of the revetment. The individual elements of the falling apron proceed down the developing scour hole and cover the scour slope, forming a continuous carpet on the river bed (see also Fig. 5.2-1).

Fig. 5.3-4: Random placed CC-bocks (falling apron)

An adequate quantity of CC-blocks has to be provided to ensure a complete protection of the scour face. The required quantity depends primarily on the depth and slope of a scour, which can be estimated at 1V : 2H in case of a fully developed equilibrium scour and a stabilized slope. It is recommended to keep a sufficient reserve of material, especially in presence of cohesive soil layers, where the slope might develop extremely irregular. At the river-sided

Y32

edge of the falling apron, where strongest flow attack can be assumed, the largest block sizes should be used.

This type of toe protection works quite well at rivers with predominant non-cohesive material. However, in case of cohesive bed or bank material or alternate layers of non-cohesive and cohesive soil, the loose material tends to fall directly onto the bottom of the scour hole, leaving some areas of the scour slope unprotected. In this situation, it is recommended to use mattresses filled with stones or boulders (Fig. 5.3-5) or cable connected block mattresses (Fig. 5.3-6). Therefore, for standardized structures a redundant system, combining both, loose protection elements (falling apron) and interconnected protection units (launching apron) are suggested.

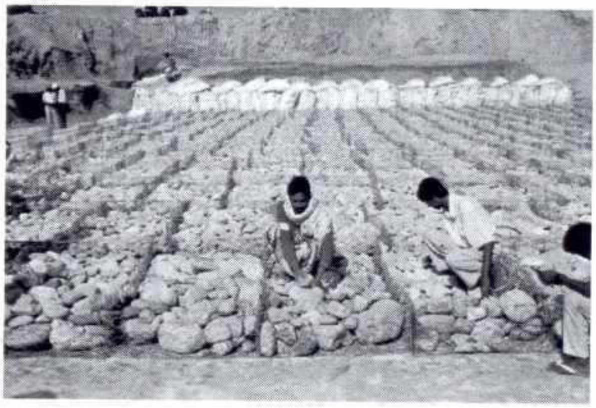

Fig. 5.3-5: Filling of wire mesh mattresses with boulders (launching apron)

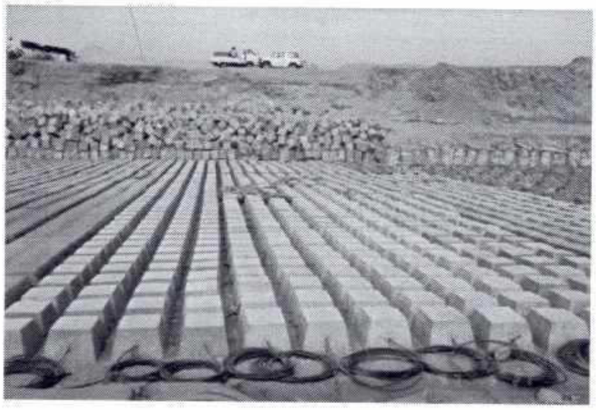

Fig. 5.3-6: Cable connected in-situ built concrete block mattress (launching apron)

5.4 DESIGN OF PROTECTIVE LAYERS

The methods given in the following Sections 5.4.1 and 5.4.2 can be used for dimensioning the protective layer units of the upper revetment cover layer and falling and launching aprons, respectively. In the given formulae loads from current and waves are considered, from which the larger result must be used for dimensioning of the protective layers. For stone filled mattress systems generally both, the mattress thickness and the nominal size of the mattress filling must be determined. For more specific information on the computation methods, refer to Chapter 8.

5.4.1 Current Attack

The comparison of different methods regarding the calculation of unit dimensions of revetment cover layers and toe protections (e.g. PIANC, 1987, Pilarczyk, 1990, Escarameia, 1992, FAP21, 1993) show only marginal deviations within the range of application for the rivers of Bangladesh. Therefore, the widely used Pilarczyk method (1990) may be recommended, because it includes the turbulence intensity by an empirical coefficient (yet merely in a very rough and qualitative way), still keeping a certain practical simplicity. It was initially developed from laboratory and field tests mainly on rip-rap, but, introducing coefficients considering the specific properties of different protection layer materials, also allows for the application on other types of revetments. The general formula for the design against current loads is given by

$$D_n \geq \frac{0.035 \cdot \bar{u}^2}{\Delta_m \cdot 2g} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{K_S \cdot \psi_{cr}} \qquad \text{(Eq. 5.4-1)}$$

where

D_n	(m)	characteristic size of the revetment cover layer material (single unit size for loose elements, thickness of mattress systems)
$\bar{u}$	(m/s)	depth averaged flow velocity; if replaced by $u_b = 0.6\,\bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
Δ_m	(-)	relative density of submerged material = $(\rho_s - \rho_w) / \rho_w$
g	(m/s²)	acceleration due to gravity (= 9.81)
ϕ_{SC}	(-)	stability factor for current
ψ_{cr}	(-)	critical shear stress parameter
K_t	(-)	turbulence factor
K_h	(-)	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2 \cdot \left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	(-)	bank normal slope factor $K_S = \sqrt{1-\left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for Bangladesh rivers and a conservative assumption
α	(°)	slope angle of bank or structure
ε_s	(°)	angle of repose considering the material specific internal friction

For calculation of the required thickness of stone or brick filled mattress systems or other interconnected units the relative density (1-n) Δ_m, considering the volume of the voids between the individual filling elements, must be applied instead of Δ_m. The percentage of voids in the mattress fill and between interconnected blocks can be estimated to

- $n = 0.4$ for stone filled mattresses
- $n = 0.15$ for brick filled mattresses
- $n = 0.1\text{-}0.3$ for cable connected block mattresses

The minimum thickness of stone filled mattresses should not be chosen smaller than 1.8 D_{50} (with $D_{50} = D_n/0.85$) or than 15 cm. Besides the stability of the whole mattress, the weight of the individual stones should be sufficient to prevent from excessive movement and thus loads on the wire mesh material. The required nominal diameter D_{50} of the filling material can also be calculated by Eq. 5.4-1, taking the respective stability coefficients for mattress filling (Δ_m, ϕ_{SC}, ψ_{cr}, ε_S) into account. The recommended grading range of stone or rock material for rip-rap revetments and mattress filling, related to the W_{50}-value (defined by $W_{50} = (D_{50})^3/\rho_s$) is given in Fig. 5.4-1. The minimum size of the stones must be larger as compared to the width of the wire mesh material.

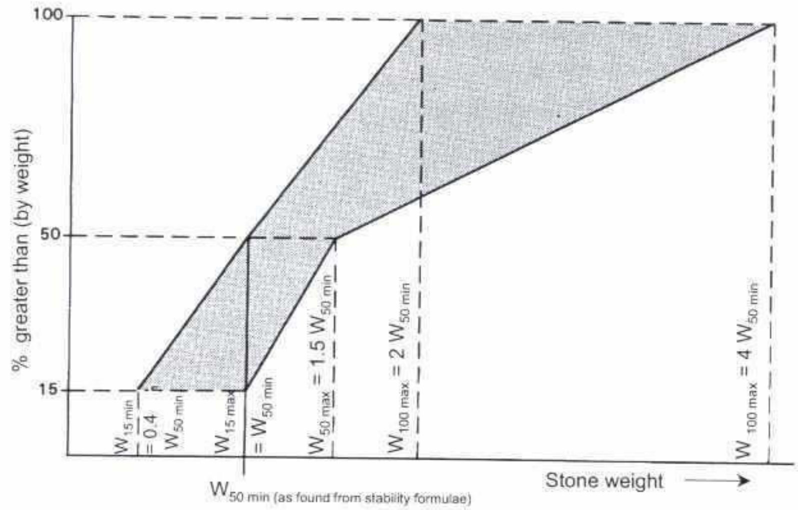

Fig. 5.4-1: Recommended grading range of stone filling for rip-rap and mattress systems (adapted from PIANC, 1987)

The range of the other coefficients used in Eq. 5.4-1 is given in the following tables. The proposed type of protective layer element/ material and the suggested assumption regarding the existing design conditions for structure categories SC2 and SC3 are marked with grey shading. The decision on the cover elements recommended for the standardized structures do not base on stability reasons only, but consider also construction constraints, available quality and economic aspects as discussed in Chapter 2. Nevertheless, if certain materials or required equipment is not at hand, alternative solutions must be chosen.

Turbulence intensity	K_t [-] Gabions, Mattresses	K_t [-] Others
Normal turbulence in rivers	1.0	1.0
Non-uniform flow with increased turbulence, mild outer bends	1.0	1.5
High turbulence, local disturbances, sharp outer bends	1.0	2.0

Table 5.4-1: Turbulence Intensity Factor K_t (Current)

Revetment type		Angle of repose ε_s [°]	Density of protection material ρ_s [kg/m^3]	Recommended application
Cover layer	Filter material			
Randomly placed, broken rip-rap and boulders	Geo-textile Granular	20 25	2600	R, FA
CC-blocks, cubical shape, randomly placed in multi-layer	Geo-textile Granular	30 35	1980	FA
CC-blocks, cubical shape, hand placed in single layer chess pattern	Geo-textile Granular	20 25	1980	R
CC-blocks, cable connected	Geo-textile Granular	20 25	1980	LA
Wire mesh mattresses	Geo-textile Granular	20 25	*)	R, LA
Gabion/mattress fillings by stones		45	2600	

*) Dependent on fill material: stones: 2600 kg/m^3, bricks:1800 kg/m^3

Table 5.4-2: Angle of repose ε_s and density ρ_s for various revetment cover layers

Revetment type	Stability factor ϕ_{SC}		Critical shear stress parameter (Shields) ψ_{cr} [-]
Cover layer	Continuous protection [-]	Exposed edges, transitions [-]	
Randomly placed, broken rip-rap and boulders	0.75	1.5	0.035
CC-blocks, cubical shape, randomly placed in multi-layer	0.80	1.50	0.035
CC-blocks, cubical shape, hand placed in single layer chess pattern	0.65	1.25	0.05
CC-blocks, (cable connected)	0.50	1.10	0.06
Wire mesh mattresses/ gabions	0.50	1.00	0.07
Gabion/mattress fillings by stones	0.75	1.5	0.09

Table 5.4-3: Stability factor ϕ_{SC} and Shields parameter ψ_{cr} for various cover materials

5.4.2 Wave Attack

Also for wave induced impacts on armouring units, several theoretical calculation methods are available for the design. Due to the various input parameters involved in the different methods a direct comparison is rather difficult. The more universal formula by Pilarczyk (1990) allows for calculation of different structure components and includes the breaker type specific dynamics of the wave impact by introducing the breaker similarity index ξ_z. The minimum dimensions for the stability of the cover material under wave attack can be determined as follows (Pilarczyk, 1990):

$$D_n \geq \frac{H_S \cdot \xi_z^b}{\Delta_m \cdot \psi_u \cdot \phi_{SW} \cdot \cos\alpha} \qquad \text{(Eq. 5.4-2)}$$

where

D_n	(m)	characteristic size of the revetment cover layer material (single unit size for loose elements, thickness of mattress systems)
H_s	(m)	significant wave height
Δ_m	(-)	relative density of submerged material = $(\rho_s - \rho_w) / \rho_w$
g	(m/s²)	acceleration due to gravity (= 9.81)
ϕ_{SW}	(-)	stability factor for wave loads
ψ_u	(-)	system specific stability upgrading factor
α	(°)	bank normal slope angle
ξ_z	(-)	wave similarity parameter = $\tan\alpha \cdot \frac{1.25 \cdot T_m}{\sqrt{H_S}}$
T_m	(s)	mean wave period
b	(-)	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

The wave similarity parameter determines the type of wave breaking, which is decisive for the actual wave impact. The formula is restricted to values $\xi_z < 3$ and $\cot\alpha \geq 2$, i.e. to plunging breakers, which generate high local pressure heads. Otherwise overestimation of the unit size is likely, because the dynamics of the breaking process are diminishing.

As for the design against current attack, the required thickness of stone or brick filled mattress systems must be calculated on basis of the relative density $(1\text{-}n)\ \Delta_m$, considering the volume of the voids between the individual filling elements. Values for n are given in Section 5.4.1. The nominal dimensions of the fill material can also be computed by Eq. 5.4-2, taking the respective stability coefficients for mattress filling (Δ_m, ϕ_{SW}, ψ_u) into account. The minimum thickness of the mattress as a unit should be larger than 1.8 D_n.

The material and armour layer unit specific coefficients to be applied for the design against wave attack are summarized in Table 5.4-4.

Revetment type	Stability factor for incipient motion ϕ_{SW} [-]	Stability upgrading factor ψ_u [-]	Interaction coefficient b [-]
Randomly placed, broken rip-rap and boulders	2.25 – 3.00	1.0 – 1.33	0.50
CC-blocks, cubical shape, randomly placed in multi-layer	2.25 – 3.00	1.33 – 1.50	0.50
CC-blocks, cubical shape, hand placed in single layer chess pattern	2.25	1.50	0.67
CC-blocks, (cable connected)	2.25	1.80	0.67
Wire mesh mattresses/ gabions	2.25	2.50	0.50
Gabion/mattress fillings by stones	2.25	2.50	0.50

Table 5.4-4: Coefficients for the design of various cover materials against wave attack

5.5 DESIGN OF FILTER MATERIAL

5.5.1 Granular Filter

The properties of granular filters depend significantly on the particle size. The filter criterion relates the grading of the filter to that of the subsoil. If the filter also has got drainage function, it is necessary to check for filter uniformity to ensure that internal migration of fines does not occur. Various authors have developed minimum filter requirements. Pilarczyk (1990) defined the following criteria regarding the relation between characteristic grain sizes of the subsoil D_s and the filter D_f.

$D_{15f} < (4 \text{ to } 5) \cdot D_{85s}$ (stability criterion)
$D_{15f} > (4 \text{ to } 5) \cdot D_{15s}$ (permeability criterion)
$D_{50f} > (20 \text{ to } 25) \cdot D_{50s}$ (segregation criterion)

$$C_u = \frac{D_{50}}{D_{10}} < 10 \qquad \text{(no migration)}$$

$$C_u = \frac{D_{50}}{D_{10}} > 20 \qquad \text{(susceptible to migration)}$$

with

C_u (-) coefficient of uniformity
D_x (mm) diameter according to x % undersize by mass taken from grain size distribution

To achieve the required filter characteristics it might be necessary to use more than one granular layer. In that case the filter has to be built in successively coarser layers starting from the underlying soil. The first layer must retain the base material, whereas the outer layer must be stable against the revetment armour layer. The minimum thickness of any granular filter is normally taken as 2 to 3 times the maximum particle size for each layer, maintain a minimum overall thickness of the granular filter of 150 mm. Wherever practicable, the granular material must be carefully compacted to minimize settlements.

5.5.2 Geo-textile Filters

The main design parameters for geo-textile filters are the retention criterion and the permeability criterion, which define the capability of the material to retain the existent sub-soil without clogging and to allow unhindered water transport trough the membrane. Besides the required functional characteristics of the geo-textile in context with the existing sub-soil properties, certain stability standards must be considered, which have to be defined with respect to the planned use and which might have further implications on the construction techniques to be employed. Specific properties of geo-textiles are available from product sheets of the respective manufacturers. In Chapter 8 minimum standards of geo-textiles recommended for different segments of standardized structures are given, which were defined following the PIANC method (1987). The PIANC design procedure involves the following steps (Fig. 5.5-1):

(1) Determination of the range of the grain-size distribution

The grain-size distribution curve must be determined following international standard regulations, to allow for calculation of the various design parameters. Due the fact, that the filter characteristics of geo-textile are mainly influenced by the fine compartment of the grain-size distribution (grading curve), the PIANC-method categorizes the soil by the screen fraction smaller than 0.06 mm grain size. Typical grain-size distributions for different soil categories A, B and C are given in Fig. 5.5-2.

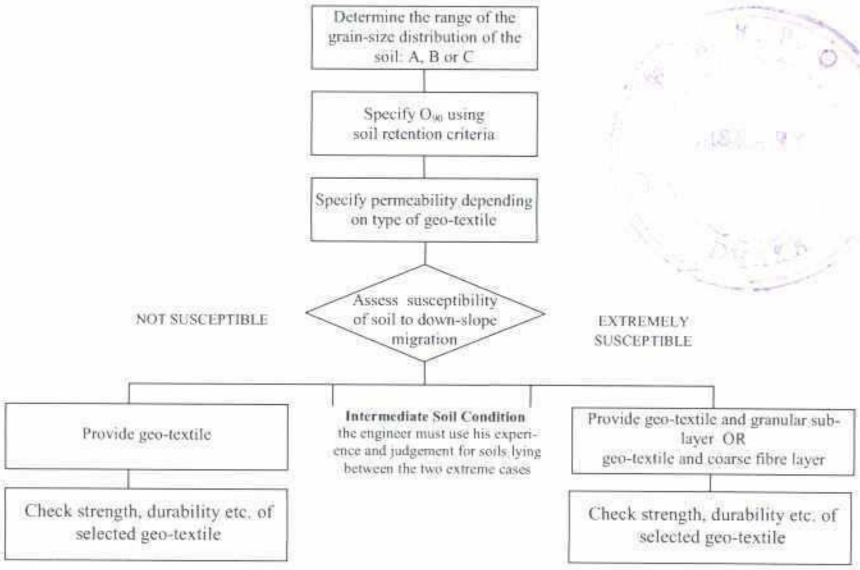

Fig. 5.5-1: Design procedure for a geo-textile filter (adopted from PIANC, 1987)

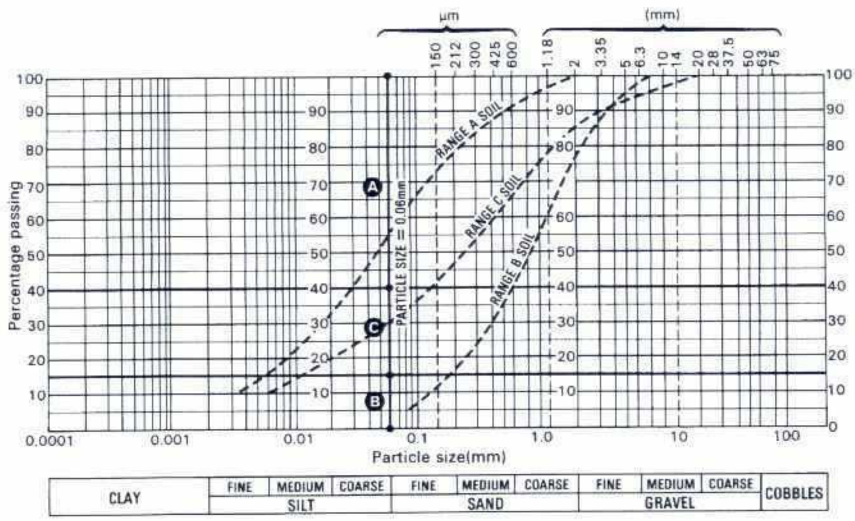

Fig. 5.5-2: Typical grain-size distributions for different soil categories (PIANC, 1987)

The soil categories for the geo-textile filter design are classified as follows (Fig. 5.5-2):

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

(2) Design for soil retention

The capacity of a geo-textile in terms of soil retention is characterized by the effective opening size O_{90}, which is defined by the equivalent diameter of a grain fraction which is retained to 90 % by the filter mat in a sieving test. This value is normally provided in the product information sheet provided by the manufacturer. The soil retention is strongly influenced by the dynamics of the impact, therefore different regulations are given for moderate stationary current and for potential highly dynamic hydraulic loads (e.g. wave impacts). The minimum requirements for the geo-textile filter to be considered for dynamic load conditions, characterized by high turbulent flow and wave attack, are specified in Table 5.5-1. The given retention criteria are only applicable for O_{90} values determined by the wet sieving analysis as defined by Swiss standard SN640550.

Grain size range	Retention criteria
A (amount of fines $\leq$ 0.06 mm larger than 40 %)	$O_{90} < d_{90}$ *) $< 10\ d_{50}$ < 0.3 mm
B (amount of fines $\leq$ 0.06 mm smaller than 15 %)	$O_{90} < 1.5\ d_{10}\sqrt{C_u}$ $< d_{50}$ < 0.5 mm
C (fines $\leq$ 0.06 mm between 15% to 40%)	As range B

*) if the soil exhibits long-term stable cohesion, then $O_{90} < 2\ d_{90}$ is applicable

Table 5.5-1: Soil retention criteria (adopted from PIANC, 1987)

(3) Design for permeability

The geo-textile filter must maintain a long-term permeability equal or larger than that of the prevailing soil. Shortly after installation a reduction in the fabric permeability due to clogging and blocking will occur, which depends on the pore structure and thickness of the material as well as on the grain structure of the soil. In general, the permeability of the geo-textile material is acceptable if

$$\eta \cdot k_g \geq k_s \qquad \text{(Eq. 5.5-1)}$$

with

- η (-) material specific reduction factor
- k_g (m/s) permeability of the geo-textile
- k_s (m/s) permeability of the soil

If k_s is not available from laboratory tests, it can be approximated by the empirical relation (Hazen, in Tomlinson, 1996):

$$k = 0.0116 \cdot D_{10}^2 \qquad \text{(Eq. 5.5-2)}$$

The reduction factor η for needle-punched and other non-woven fabrics thicker than 2 mm (measured at a normal stress of 2 kN/m^2) is defined as a constant:

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

(4) Identification of soils susceptible to down-slope migration

The migration of soil particles underneath the geo-textile filter layer by gravity induced down slope water transport (e.g. rain water or wave run-down) is apparent particularly in soils with fine, but non cohesive material (hence, mainly grading range A and B). Therefore, a distinction between soils susceptible to down-slope migration and those not susceptible can be related to grain size, permeability and cohesion of the existing soil sub-layer (revetment core). Soils susceptible to down-slope migration are in general characterized by the following conditions:

grain size	:	a considerable amount of fine material with $d < 0.06$ mm and more than 50 % of the soil ranging between $0.02 \text{ mm} < d < 0.1 \text{ mm}$
$C_U = d_{60}/d_{10} < 15$	:	coefficient of uniformity (conservative estimate based on practical experience; for larger C_U values normally a secondary filter will establish)
$I_p = w_L - w_p < 0.15$	:	limited value for plasticity index. The plasticity index is defined as the difference between the moisture content at liquid limit (w_L) and the moisture content at plastic limit (w_p). If I_p is not known, the following criterion which relates the weight of the clay fraction to the weight of the silt fraction may be used for preliminary design:

$$\frac{W_{(d\,<\,0.002\,mm)}}{W_{(0.002\,mm\,<\,d\,<\,0.06\,mm)}} < 0.5$$

The above given conditions allow only for a rough estimation regarding the risk of down-slope migration processes at certain subsoil properties. For consideration of additional influencing conditions (see also paragraph (5) below) and site specific factors, an experienced soil mechanics engineer should be consulted.

(5) Counter measures against down-slope migration

Down-slope migration of soil particles underneath a geo-textile filter can be prevented by incorporating a granular sub-layer between the geo-textile and the cover layer (thickness of approx. 300 mm), which should be fine enough to provide an adequate damping effect and coarse enough to be retained by the cover layer.

Furthermore, the surface of the soil underneath a geo-textile can be stabilized by special composite geo-textiles, providing a second, relatively thick layer (minimum of about 5 mm) of coarse fibres attached to the back of the geo-textile filter. The required physical properties of the synthetic coarse layer are dependent on the soil grading range (Table 5.5-2). As an alternative to increase the slope stability, a heavy weight cover layer, which is reducing potential uplift forces caused by excess pore water pressure, can be applied.

202

Characteristics of coarse layer	Range A Grading curve	Range B Grading curve
effective opening size O_{90}	$0.3 < O_{90} < 1.5$ mm	$0.5 < O_{90} < 2.0$ mm
thickness of geo-textile t_{gg}	$5 < t_{gg} < 15$ mm	$5 < t_{gg} < 20$ mm

Table 5.5-2: Minimum standards of the coarse layer (PIANC, 1987)

5.6 DESIGN OF TOE PROTECTION

Taking into consideration the possibility of prevailing non-cohesive and cohesive soil materials (soil stratification), for standardized structures (SC2 and SC3) a redundant system, combining loose protection elements (falling apron) and interconnected protection units (launching apron) is recommended. The suggested protection elements are cable connected concrete blocks or stone filled cable connected wire mesh mattresses for the launching apron and loose concrete cubes for the falling apron. The actual dimensioning of falling and launching apron is interrelated, therefore the given method is based on the combined implementation of both components.

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

After completion of the construction works the toe protection must remain stable under the existing flow and wave conditions during monsoon season (water levels higher than FPL). The most important concern is, that no larger part of the material is transported in flow direction. However, the flow velocity above the falling apron (initial situation, compare Fig. 5.2-1) during monsoon season is reduced due to the relatively small water depth and the large roughness of the armour. Moreover, wave activity is assumed to show insignificant influence on the efficiency of the falling apron (slight rocking and smaller dislocation is acceptable). On the other hand, wave loads should be given more consideration, where relatively frequent wave action is prevalent (river stretches with large fetch and high wind velocities, estuaries, etc.).

Subsequent to the articulation and reformation of the material along the scour slope, the elements must be able to persist shear stresses of the existing current. At this location the depth averaged design flow velocity u_{toe} can be approximated by the hydraulic design velocity $\bar{u}_{des}$. For that situation wave impacts are rather unlikely.

Besides these remarks concerning function and safety aspects, the dimensioning of the proposed scour protection blankets is based on the following assumptions regarding the final (articulated) situation:

- a berm (safety margin) remains in front of the upper revetment (between toe of the embankment slope and the assumed edge of the maximum (equilibrium) scour);
- the profile of the stabilized scour hole will develop at a slope of about 1V : 2H;
- a theoretical block layer of 1.5 D_n (without voids) is sufficient to allow for the coverage and stabilization of the scour slope.

5.6.1 Dimensions of Launching Aprons

The design of the recommended mattress and cable connected concrete block systems can be derived from the formulae given regarding current and wave attack in Sections 5.4.1 and 5.4.2. It is recommended to place the launching apron units on a heavy geo-textile as defined in Section 8.4. Both, the geo-textile mats and the interconnected cover units need a sufficient anchoring at the toe of the upper revetment, i.e. a dead man preferably made from steel, with a minimum length of 4 to 5 m.

The required cable dimensions and properties for cable connected concrete block systems are given in Chapter 8 and the Design Plates. The width of the launching apron should be chosen to W_{LA}= 20m for standardized protection structures of category SC2 and SC3.

5.6.2 Effective Volume and Dimensions of Falling Aprons

The required size of individual CC-blocks can be computed by the formulae given in Section 5.4. It is recommended to apply a minimum block size D_n = 0.3m. Following the assumptions made for the calculation of the minimum volume required to cover the expected equilibrium (maximum) scour hole, a geometrical solution based on the scour profile and multiplied by a safety factor can be applied. The required volume V_{FA} of a scour protection blanket per metre bank line can be estimated to

$$V_{FA} = 1.5 \cdot D_n \cdot \sqrt{5} \cdot y_{BL} \cdot C_{FA} \qquad \text{(Eq. 5.6-1)}$$

with

V_{FA}	(m^3/m)	Volume of falling apron per linear metre protected length
D_n	(m)	block size (1.5 D_n is the proposed layer thickness after scouring without voids)
y_{BL}	(m)	vertical distance between base level of falling apron at time of construction and deepest point of the expected design scour hole
C_{FA}	(-)	flow attack coefficient : 1.50 (moderate flow attack) 1.75 (strong flow attack)

The term $\sqrt{5} \cdot y_{BL}$ describes the simplified area of the landward scour profile in m^2/ linear metre, assuming a 1V in 2H stabilized scour slope (see also Section 5.3.4). The recommended construction base level of the falling apron is set at SLW +0.5m, in case this precondition is modified due to other site specific reasons, it must be taken into account in the computation of the required material quantity. The design of a typical falling apron design is schematically shown in Fig. 5.6-1.

The width of the falling apron should be chosen to 70 to 100% of the distance y_{BL} between expected deepest point of the scour hole and the level of the scour protection elements during construction.

$$W_{FA} = 0.7 \text{ to } 1.0 \cdot y_{BL} \qquad \text{(Eq. 5.6-2)}$$

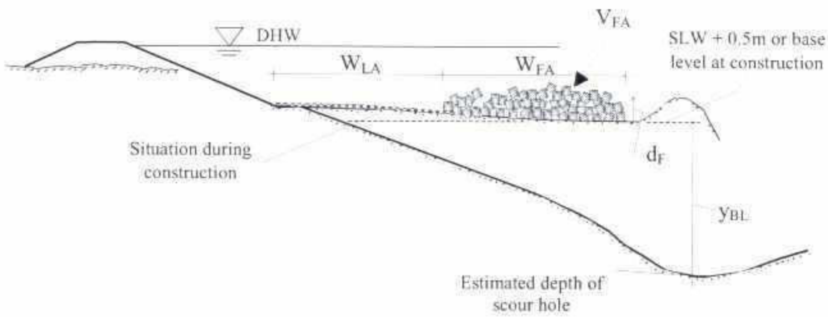

Fig. 5.6-1: Geometric properties of a typical falling apron for standardized revetments (schematic)

Under the given assumptions, the theoretical thickness of the falling apron d_{FA} (without voids) follows from Eq. 5.6-1 and Eq. 5.6-2 to approximately

$$d_{FA} = 5.0 \text{ to } 8.5 \cdot D_n \qquad \text{(Eq. 5.6-3)}$$

The actual thickness of the falling apron material during construction is larger, because of the voids between the individual CC-blocks. Nevertheless, it is recommended to dump the blocks to form a mound, rather than to place the elements in rows or columns, because it is assumed that the articulation of the falling apron is improved. The minimum thickness of the falling apron should be larger than d_{FA}= 3 D_n, whereas the maximum thickness is dependent on the construction technique used (manual labour or mechanically placed) and the maximum tolerable load on the existing sub-soil.

To control the quantity of material to be provided for the designed falling apron width W_{FA}, the number of blocks per linear metre of protected length can be calculated by

$$n = \frac{V_{FA}}{D_n^3} \qquad \text{(Eq. 5.6-4)}$$

In case, reliable construction techniques for under water placement of toe protection material during high flow velocity conditions are at hand in the country in future, the same approach as given in this Section can be used. The under water construction has got certain advantages due to the smaller flood plain area involved for the structure implementation. Because of the considerable length of the under water protective blanket (Fig. 5.6-2), which is expected to remain stable at the design hydraulic and morphologic boundary conditions, the material required for the actual falling apron is much smaller. On the other hand the total amount of placed protective materials in both solutions is about corresponding.

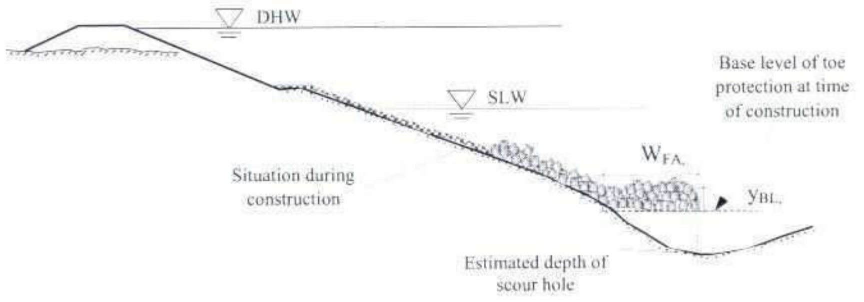

Fig. 5.6-2: Toe protection of a revetment for under water construction (schematic)

207

6 DESIGN OF PERMEABLE GROYNES

6.1 GENERAL ASPECTS

The behaviour of individual groynes or groyne fields has been subject to numerous research studies in field and laboratory. At present, still many different types are built all over the world (refer to Chapter 2), and optimal properties seem to depend to a large extent on the particular site conditions. Normally, a conceptual design study has to be carried out to find a solution, which is expected to be optimal. In general, groynes are built at straight sections or at concave bends (outer bank) of meandering or braided rivers. A groyne field consists typically of a series of impermeable or permeable or a combination of impermeable and permeable groynes. Only in exceptional cases groynes are also built as single structures.

A brief multi-criteria assessment of advantages and disadvantages regarding different groyne types is given in Table 6.1-1. Considering the experience with the very particular situation at the major rivers of Bangladesh, certain restrictions to the layout seem appropriate to cope with the vulnerability against scouring of the river bed and with the prevailing huge sediment transport rates. In case impermeable groynes are constructed, a substantial volume of toe protection is required to stabilize the structure induced scour, especially at the head of the impermeable groyne. In contrast to this, an advantage of permeable pile structures is, that they can be installed partly under water, which reduces the space required for construction. From this point of view, the implementation of permeable groynes is recommended for bank protection measures. Permeable structures can be categorized as active measures, because the impulse flux of the water mass is reduced by internal and external friction, when passing through the structure gaps. A typical layout of a permeable groyne is shown in Fig. 6.1-1 and Fig. 6.1-2.

Permeable groynes generally consist of one or several rows of timber piles, steel piles or reinforced concrete piles, which must be designed to resist the expected hydraulic loads and additional forces induced by floating debris and ship impacts. Nevertheless, ship impacts are not considered in the design methods given below for structure categories SC 2 and SC 3. Due to the long lever arm, these exceptional forces would result in substantial overdesign. At rivers, where more frequent vessel traffic is apparent, additional reinforcement and fendering of the most river-sided piles is required. The application of timber piles is restricted to rather small channels with low water depth (structure category SC 1) and are not discussed in this handbook.

Subsoil investigations and determination of the characteristics of the different layers are part of the preparatory works. Soil parameters such as specific weight, angle of internal friction and degree of cohesion are required for the determination of the embedded length and properties of the piles. The type of soil and its density are also decisive for the selection of a suitable piling techniques.

In case an impermeable groyne section at the structure root (to be constructed on the dry flood plain) is required, the transition between impermeable and permeable sections have to be thoroughly designed especially when higher flow velocities are expected in this area. A gradual slope of 1V:5H or less at the head of the impermeable section and an adequate scour protection blanket should be provided, because flow separation might result in severe large eddy formation and strong return currents.

Multi-Criteria-Aspect	Impermeable Groynes			Permeable Groynes		
	Earth Dams	Cofferdams: Steel sheet piles	Cofferdams: Concrete sheet piles	Steel Piles	Concrete piles: Pre cast	Concrete piles: In situ cast
1. Susceptibility against impact loads						
Vulnerable to unexpected hydraulic loads	yes	yes	Yes	no	no	no
Vulnerable to unexpected scouring	limited [1]	limited [1]	limited [1]	no	no	yes
smooth transition between impermeable and permeable section	limited	limited	not applicable	n.a.	n.a.	n.a.
2. Local availability of materials and skill						
Suitable for Bangladesh contractors	available	available	available	yes	yes	yes
Availability of material in Bangladesh	yes	yes	Yes	producible	producible	available
Suitable for strategic material stock-piling	yes [2]	yes [2]	yes [2]	yes	yes	yes (basic materials)
Specialised skills	no	sheet pile installation		pile fabrication, welding and large diameter pile installation		no
3. Possible constraints during construction						
Installation on the flood plain	yes	yes	Yes	limited by possible total driving depth		yes
Installation into the river under moderate flow conditions	no	limited	Limited	yes	yes	no
Site mobilisation time (minimum period required prior to actual start of the construction)	short	short	Short	depends on availability of piling equipment		short
Installation / Construction time	moderate	moderate	moderate	short	short	moderate
4. Adaptability after completion						
Adaptability to specific local conditions (e.g. for repair or to accommodate river response)	easy	complicated	complicated	easy	easy	impossible, piles must be replaced
5. Socio-economic value						
Job opportunities and participation of local population during construction	high	good	Good	low	low	moderate

Table 6.1-1: Comparison of different groyne types for bank erosion protection measures

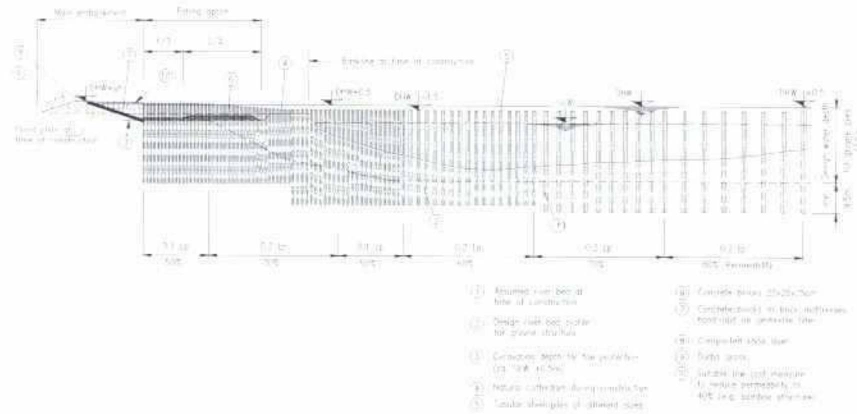

Fig. 6.1-1: Typical permeable groyne (cross-section)

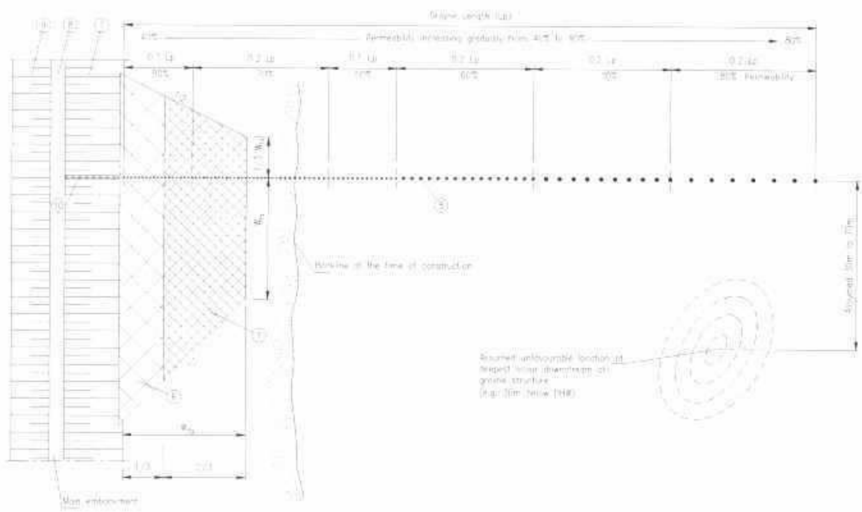

Fig. 6.1-2: Typical permeable groyne (plan view)

6.1.1 Overall Length, Alignment and Terminations of Groyne Fields

The general implications regarding the overall length, alignment and terminations are very similar to those of revetments. However, the structural layout of a groyne field bears a substantially larger number of possible variations of different structure components due to the strong interference between the most important characteristic properties (porosity, spacing, individual length, etc.). As for revetment structures, a connection between groyne field and the main flood embankment is mandatory unless the groynes are directly connected to it.

The standard layout of a groyne field consists of a central section with a series of at least 3 groynes similar in composition and length plus an upstream and downstream termination with shorter groynes (with decreasing length), in order to cope with a possibly developing upstream embayment and to smooth out the transition between protected and unprotected reach downstream from the groyne field (see Fig. 6.1-3). At the same time, shorter upstream groynes restrict the hydraulic loads acting on the longer groynes in the central section.

In some cases the local situation might require specific solutions.

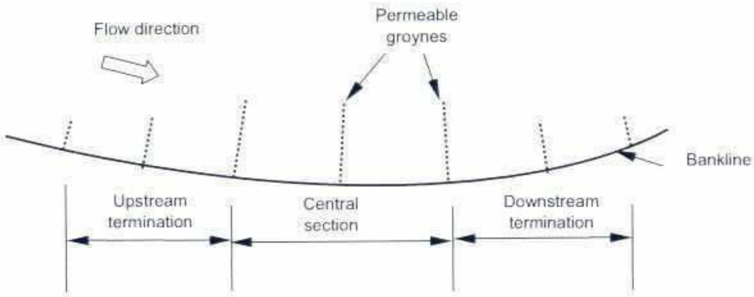

Fig. 6.1-3: Schematic layout of permeable groyne field

6.1.2 Permeability of Groynes

The permeability P of a groyne is defined by the ratio of open (non-blocked) area to the total area, which can be expressed by the quotient of internal width s and the distance e between the axis of two adjacent piles ($P = s/e$, see Fig. 6.1-4). In general, keeping the same target permeability, larger pile diameters are considered more economical as compared to smaller diameters because the material quantity and the construction time is reduced. Physical model tests substantiated the fact, that the functional efficiency of single-pile-row-groynes using relatively large pile diameters (1.0 m to 1.4 m) is similar to twin- or triple-pile-row-groynes with smaller pile diameters of 0.5 m to 0.7 m for identical permeability of both groyne structures. A minimum permeability of $P = 0.5$ (50%) should be kept for constructional reasons (increased pile-driving resistance). If lower values are required, the groyne might preferably be constructed in two neighbouring rows in the respective parts.

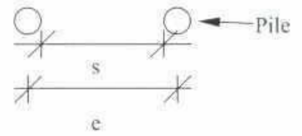

Fig. 6.1-4: Definition of groyne permeability

The optimum permeability of a groyne is dependent on various boundary parameters (flow velocities, local turbulence, grain size distribution, etc.) and is strongly interrelated with structural conditions of the groyne field (number and length of groynes, spacing in flow direction). The design of the groyne must allow for an efficient reduction of flow velocities along the groyne axis, to prevent the river bank from undermining. This calls for a low permeability. On the other hand, with increasing the permeability of a groyne, the effects related to flow separation and large eddy generation (followed by return currents) and thus, local scouring at the groyne head, are considerably reduced. Furthermore, construction costs can be limited.

In order to achieve a most gradual transition from the non-blocked to the partially blocked river cross-section and to create a more or less constant flow resistance from the head to the root (allowing for bank parallel flow lines), it is recommended to decrease the permeability of the groyne towards the embankment. The following pile arrangement (Table 6.1-2) is recommended for standardized groyne structures (SC2 and SC3). For the ease of construction, the total groyne length L_G may be divided into 4 sections of equal length with a constant permeability each.

Section	Permeability (%)
First pile near the embankment (groyne root)	
0.00 to 0.25 L_G	50
0.25 to 0.50 L_G	60
0.50 to 0.75 L_G	70
0.75 to 1.00 L_G	80
last pile in the river (groyne head)	

Table 6.1-2: Recommended pile arrangement of a permeable groyne

Also groynes with randomly distributed permeability along their axis have been executed in the past and have shown good results. However, it has to be kept in mind, that an effective reduction of the flow velocities near the bank is a main goal of groynes. Therefore, large permeabilities ($P > 60$ %) in this area should be avoided. With respect to optimise the permeability distribution along the groyne axis further systematic field and model studies are strongly recommended.

6.1.3 Orientation of Groynes

The deviation of the streamlines is mainly dependent on the structure induced blockage of the flow cross-section, which can be described by the length and permeability of the structure in relation to the geometrical channel properties. Consequently, for permeable groynes the actual shape (jockey head, etc.) of the groyne and the angle between groyne axis and bank line is of rather small importance as compared to impermeable spurs. Taking also into account the expected variations in the approach direction of the critical channel during the subsequent years after construction, it is economical (because $L_G = L_{G,eff}$, refer to Section 6.2.1) and appropriate from the hydraulic point of view, to implement the individual permeable groynes in bank normal direction (90°).

6.1.4 Crest Level

In order to restrict the hydraulic loads (currents, waves and floating debris) at higher water levels, groynes may be designed as submerged structures (during peak discharges). This would increase the durability of the structure and require considerably shorter embedment lengths. Nevertheless, slightly higher flow velocities near the bank line would have to be expected in that case.

Floating debris transported by the river, dominated by clusters of water hyacinths, banana trunks and other material can become significant for the structure stability. For example, a maximum area of floating debris of about 10,000 m^2 with a thickness of up to 2 m is expected to occur frequently in the Jamuna river. The largest amount of floating debris can be expected during or shortly after peak flows, because flood plain areas upstream from the groynes are getting inundated, washing away parts of the existing vegetation and other material. In general, the thickness of the layer of floating debris increases as the flow velocity in the channel increases.

Floating debris trapped by the piles will influence the blockage and subsequently the scour development downstream from the groyne. To reduce the influence of floating debris at high water levels, the groynes can be designed with a negative freeboard, i.e., they act as slightly submerged groynes, allowing the debris to float just above the crest of the piles. A compromise is given by designing a variable crest level along the groyne axis to keep the functional efficiency and reduce negative effects by trapped floating debris. However, towards the groyne root, the crest level of piles should increase to meet the elevation of the embankment. In case of partly or completely submerged groynes (during high flood level) the installation of navigation signals at the groyne's head is obligatory.

6.1.5 Main Flood and Secondary Embankments

Dependent on the local situation, the planned groyne field should be built directly in front of the BWDB main flood embankment, or in front of a new secondary embankment, which is restricted to the overall length of the groyne filed (including terminations). Analogous to revetment structures, the secondary embankment must be connected to the main flood embankment by an earth dam. Due to the fact, that, in contrast to flood embankments, a groyne field should protect against bank erosion, some additional preparatory works and minimum standards must be kept. A protection layer at the river sided slope is obligatory.

Minimum standards for embankments at groyne fields should be as follows:

- top level of the earth dam according to design high water level (DHW) plus freeboard
- crest width ≥ 4.25 m
- sides sloped at 1 in 3, at river-side and land-side
- suitable fill material from borrow pits with maximum 5 % silt content for material with steep grain size distribution curve, or of 10 % for material with flat curves
- layer-wise compaction of earth dam fill to achieve a sufficient degree of density

Fill material is normally being obtained from borrow pits opened along the embankment to be built. If planned landward the borrow pits should be arranged maintaining at least a 5 m wide berm between the planned toe of the main embankment and the upper edge of the pit (Fig. 6.1-5). The arrangement of borrow pits at the river side of a new embankment should be restricted to the excavation level of the planned toe protection.

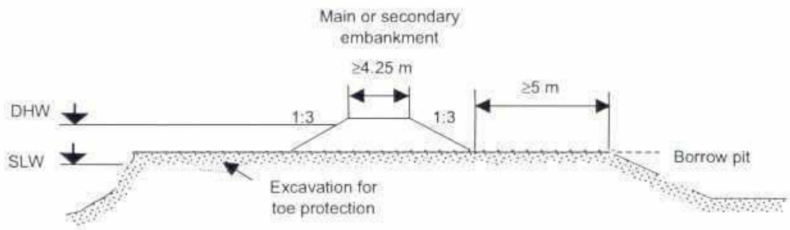

Fig. 6.1-5: Typical cross-section of a borrow pit for construction of the main embankment

The design of the river-sided slope and toe protection of the main embankment can be determined with the methods described in Chapter 5, but taking into account the reduction of flow velocities along the groyne axis.

6.2 DESIGN OF INDIVIDUAL GROYNES

6.2.1 Groyne Length

The effective length of a groyne L_G is defined as the length projected on a theoretical line perpendicular to the river bank. For orthogonal groynes, the effective length and the linear groyne length are identical. The main design aspect regarding the minimum effective groyne length L_G is to sustain the embankment stability with regard to the developing scour hole downstream from the groyne head.

286

Assuming a natural scour slope (dependent on the existing subsoil) developing from the deepest point of the scour hole towards the bankline, the minimum groyne length can be calculated according to Fig. 6.2-1. The actual location of the scour hole will shift towards the embankment, in case oblique flow attack is prevalent. For structure categories SC 2 and SC 3 a most unfavourable approach angle of $\theta = 45°$ at braided rivers is considered. For larger θ - values modifications of the structural layout are essential. Additionally a safety margin ΔL of about 10m at the toe of the embankment should be kept.

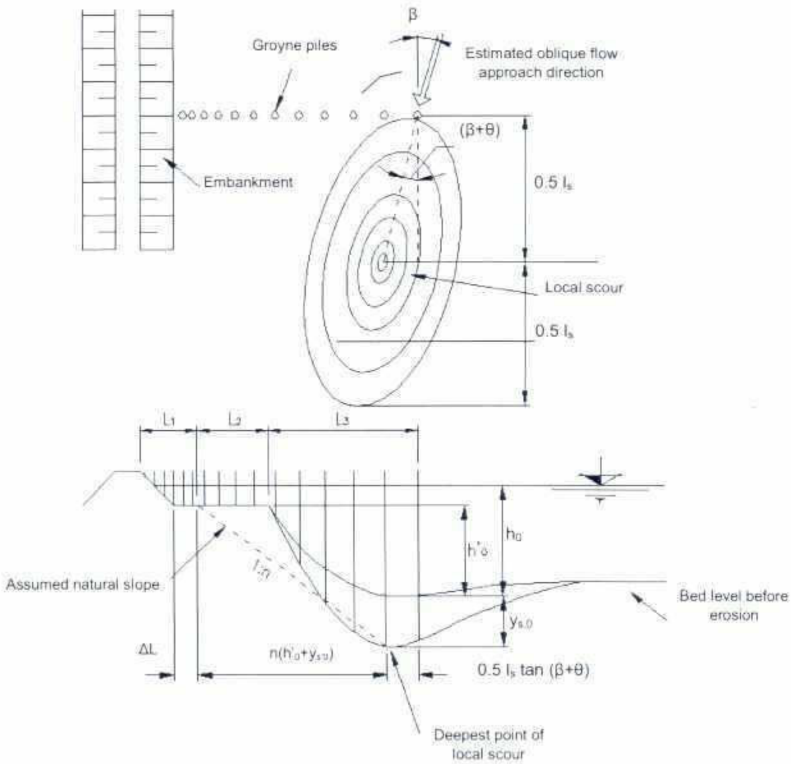

Fig. 6.2-1: Assumed scour development at the groyne head

With this, the minimum effective length of a permeable groyne in the central section of a groyne field is defined by (Eq. 6.2-1) whereas an upper limit is given by (Eq. 6.2-2) to reduce possible negative impacts at the opposite bank line. Both formulae were developed from analysis of model tests and field observations at the Jamuna. However, the values obtained by the formulae are recommendations and it might be necessary to deviate from it.

$$L_G \geq \Delta L + n \cdot (h_0' + y_{s,0}) + 0.5 \cdot l_s \cdot \tan(\beta + \theta) \qquad \text{(Eq. 6.2-1)}$$

and

$$L_G \leq 0.2 \ to \ 0.4 \cdot B_{ch} \qquad \text{(Eq. 6.2-2)}$$

with

L_G	(m)	effective length of groyne (perpendicular to the embankment)
ΔL	(m)	safety margin (minimum $\Delta L = 10$ m)
n	(-)	cotangent of natural slope of the bed material 1(V) : n(H)
h_0'	(m)	water depth at the thalweg referred to FPL
$y_{s,0}$	(m)	maximum total scour depth related to the thalweg of the undisturbed river bed
l_s	(m)	length of scour hole perpendicular to the groyne axis with first estimate: $l_s = 4\ y_s$
θ	(°)	angle of flow attack between flow line and bankline
β	(°)	fictitious angle of flow separation (see Section 6.2.2)
B_{ch}	(m)	average width of the approach channel

The assumed scour slope must be smaller than the angle of repose of the bed material. Depending on the subsoil, values of 10 to 12 degrees are recommended. As a first approach, for sandy and non-cohesive soils (as prevailing at the Brahmaputra-Jamuna and other rivers of Bangladesh), n = 5.5 can be chosen (compare Chapter 4). If minor damages can be tolerated, n = 4 is acceptable. For cohesive soils (Eq. 6.2-1) leads to a rather conservative calculation of L_G.

6.2.2 Spacing of Permeable Groynes

For a series of permeable groynes a wide range of recommendations regarding their optimal spacing exists. With certain exceptions, the executed and recommended spacing range between about 1.5 to 5 times the effective groyne length. From the economical point of view, the spacing should be as large as possible. However, the efficiency of the groyne field as a whole shall not be affected by too large spacing.

A groyne can be considered as a disturbance of the flow field, which diminishes in downstream direction and finally becomes neutralized after a certain distance, called the relaxation length λ_w. The relaxation length follows from a linearized balance between the convective term and the friction term in a one-dimensional momentum equation, which is defined for a bank parallel flow as:

$$\lambda_w = c_5 \frac{C^2 h}{2g} \qquad \text{(Eq. 6.2-3)}$$

in which

λ_w (m) relaxation length
C ($m^{0.5}/s$) Chézy coefficient
g (m/s^2) acceleration due to gravity
h (m) local water depth
c_5 (-) empirical coefficient for channel properties

Values for c_5 are given in Table 6.2-1.

Channel alignment	Water depth in groyne field	Scour hole	Coefficient c_5
Bend	Deep	deep	0.85
Straight	Deep	moderate / deep	0.70
Straight	Half of main channel depth	No / moderate	0.50

Table 6.2-1: Recommended values for c_5 dependent on local situation

For impermeable groynes under bank parallel flow approach the groyne spacing SG is recommended at a value of approximately SG = 2/3 λw (e.g. Jansen et al., 1979). A factor of 2/3 has been taken into account with respect to keep the separating flow line at a certain distance from the bank line.

To allow for inclusion of potential oblique flow attack ($\theta \neq 0$) a fictitious separation angle β is defined by (Eq. 6.2-4), which is based purely on geometrical considerations as demonstrated in Fig. 6.2-2.

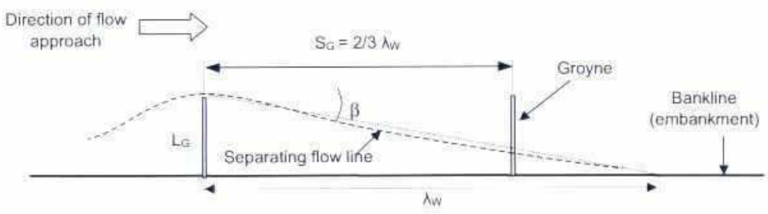

Fig. 6.2-2: Theoretical separating flow line (bank parallel flow)

$$\tan \beta = c_5 \cdot 2g \cdot \frac{L_G}{C^2 \cdot h} \qquad \text{(Eq. 6.2-4)}$$

It is obvious, that using (Eq. 6.2-4) for calculation of the relaxation length λ_w is not consistent with physics. Introducing a potential angle of oblique flow attack, the minimum spacing S_G between two adjacent groynes is given by

$$S_G = \frac{2}{3} \cdot \frac{L_G}{\tan(\theta + \beta)} \qquad \text{(Eq. 6.2-5)}$$

where

L_G (m) effective length of permeable groyne
θ (°) angle of oblique flow attack
β (°) fictitious separation angle

Downstream from permeable groynes, different physical processes influence the flow field, so that the application of the above formula is restricted. Further is necessary to determine the optimal spacing of permeable groynes.

6.3 DESIGN OF PILES

The individual piles of a groyne must resist the existent loads due to current, waves and floating debris. Furthermore, loads resulting from ship impacts or earthquakes might be of importance, but are not considered in this handbook for the design of standardized groynes.

Pile design comprises the determination of the required embedment length, which is needed to transmit the loads acting on the pile structure into the subsoil as well as the dimensioning of the required pile diameter and wall thickness.

6.3.1 Current Induced Loads

The single piles have to withstand the forces resulting from the flowing water. Due to the effect of the groynes, the flow velocities - and thereby the corresponding loads to the piles – are reduced along the groyne axis towards the embankment. From the evaluation of monitoring data and numerical calculations, a stepwise reduction of the average flow velocity as shown in Fig. 6.3-1 was derived and is recommended for the design of standardized groynes.

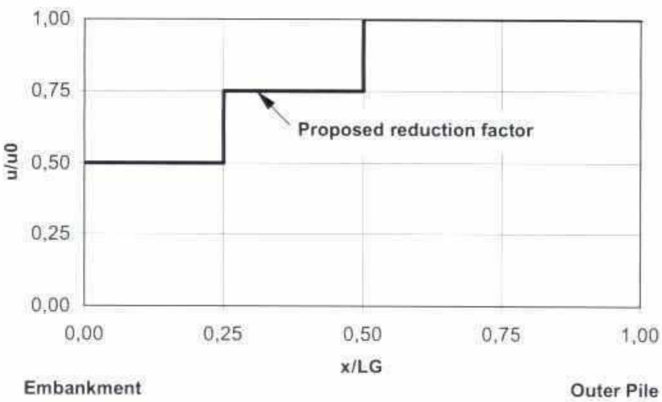

Fig. 6.3-1: Reduction of flow velocities along groyne axis (recommended)

The current induced load on a single pile can be determined by a momentum approach, based on the velocity distribution over the water depth:

$$p(z)=\int_{z=0}^{z=h} C_D \cdot 0.5 \cdot \frac{\gamma_W}{g} \cdot D \cdot u_1^2(z)dz \qquad \text{(Eq. 6.3-1)}$$

where

C_D	(-)	drag coefficient to characterize the structure fluid interaction (friction, viscosity and turbulence ($C_D = 0.7$ for circular piles)
γ_w	(kN/m^3)	density of water
g	(m/s^2)	acceleration due to gravity
D	(m)	pile diameter
$u_{1(z)}$	(m/s)	depth dependent flow velocity
z	(m)	vertical co-ordinate (water level surface: z = 0)

The flow velocity distribution over the water depth is given by a logarithmic profile (see also Chapter 4):

$$u_1(z) = u_1 \cdot \frac{\ln\left(30 \cdot \frac{d_1 - z}{k_s}\right)}{\ln\left(11 \cdot \frac{d_1}{k_s}\right)} \qquad \text{(Eq.6.3-2)}$$

where:

u_1	(m/s)	average upstream flow velocity
k_s	(-)	coefficient of roughness for the river bed

Due to narrowing of the flow cross-section between adjacent piles of a permeable groyne a local water level set up upstream from the structure occurs which is followed by a decrease in the upstream velocity. Based on the assumption of a constant water level gradient at the structure, the water level difference h_s can be calculated by the REHBOCK formula:

$$h_s = \alpha \cdot [\delta - \alpha \cdot (\delta - 1)] \cdot (0.4 + \alpha + 9 \cdot \alpha^3) \cdot \left(1 + \frac{u_2^2}{g \cdot d_2}\right) \cdot \frac{u_2^2}{2g} \qquad \text{(Eq. 6.3-3)}$$

where

h_s	(m)	water level difference at pile
α	(-)	D / c = 1-P, blockage
D	(m)	diameter of pile
c	(m)	distance between two adjacent piles
δ	(-)	shape coefficient (δ = 2.10 for round, δ = 3.80 for square piles)
d_2	(m)	undisturbed downstream design water depth
u_2	(m/s)	average undisturbed downstream velocity

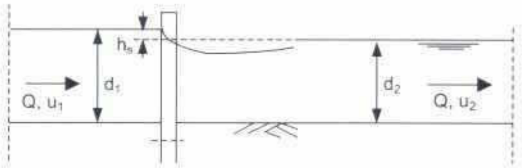

Fig. 6.3-2: Current induced water level set-up upstream of a single pile

With the simple relation $d_1 = d_2 + h_s$ and with $Q = d_1 \cdot u_1 = d_2 \cdot u_2$ (continuity equation, constant channel width) the average upstream flow velocity can be calculated

$$u_1 = \frac{u_2 \cdot d_2}{d_1} \qquad \text{(Eq. 6.3-4)}$$

6.3.2 Wave Induced Loads

The wave induced load resulting from progressive waves consists of drag forces due to velocity components and inertia forces which are generated by acceleration components of the orbital motion. Due to the fact, that the maximum of drag and inertia forces under a progressive wave occur at different wave phases (theoretical phase lag of a quarter wave length for linear waves) both load components must be computed and superimposed for different phase angles. For comparatively slender elements ($D/L \leq 0.05$), the resulting non-breaking wave load on a vertical pile can be approximated by the MORISON formula (SPM 1984):

$$p(z) = p_D(z) + p_M(z) = \beta\left(C_D \cdot \frac{1}{2} \cdot \frac{\gamma_W}{g} \cdot D \cdot u(z) \cdot |u(z)| + C_M \cdot \frac{\gamma_W}{g} \cdot \frac{D^2\pi}{4} \cdot \frac{du(z)}{dt}\right) \qquad \text{(Eq. 6.3-5)}$$

and

$$p = \int_{z=0}^{z=h} p_D(z) + p_M(z)\,dz \qquad \text{(Eq. 6.3-6)}$$

where

$p(z)$	(kN/m)	depth dependent wave force
p	(kN)	total resulting wave force on a single pile
p_D	(kN/m)	drag force per unit length of pile
p_M	(kN/m)	inertia force per unit length of pile
C_D	(-)	drag coefficient ($C_D = 0.7$)
C_M	(-)	inertia coefficient ($C_M = 2.0$ for circular piles)
g	(m/s²)	acceleration due to gravity
γ_W	(kN/m³)	density of water ($\gamma_W = 10$ kN/m³)
β	(-)	correction factor for narrow spaced piles

At alluvial rivers, velocity and acceleration components computed by linear wave theory will provide sufficient accuracy for the calculation of non-breaking wave loads on pile structures. In case of more severe wave attack is expected (estuaries, etc.) a more detailed study of the wave forces might be needed. The phase related velocity and acceleration components under a linear deep water wave are given by:

$$u = \frac{H}{2} \cdot \omega \cdot e^{k \cdot z} \cdot \cos \vartheta \qquad \text{(Eq. 6.3-7)}$$

$$\frac{du}{dt} = \frac{H}{2} \cdot \omega^2 \cdot e^{k \cdot z} \cdot \sin \vartheta \qquad \text{(Eq. 6.3-8)}$$

where

d	(m)	water depth
u	(m/s)	horizontal component of the orbital velocity
du/dt	(m/s^2)	horizontal component of the orbital acceleration
H	(m)	wave height
t	(s)	temporal co-ordinate
T	(s)	wave period
ω	(1/s)	angular wave frequency $\omega = \frac{2 \cdot \pi}{T}$
L	(m)	wave length $L = \frac{g \cdot T^2}{2 \cdot \pi}$
e	(-)	base of the natural logarithm (e = 2.718...)
k	(1/m)	wave number $k = \frac{2 \cdot \pi}{L}$
z	(m)	vertical co-ordinate (z = 0, still water table)
ϑ	(-)	phase angle $\vartheta = k \cdot x - \omega \cdot t$ (describing temporal and spatial development of a progressive wave)
x	(m)	spatial co-ordinate

The correction factors β for wave loads on narrow spaced pile clusters are adapted from EAU (1996). For a relative spacing between 2 < e/D < 4 the correction factor β can be calculated by

$$\beta = 2 - 0.25 \frac{e}{D} \qquad \text{(Eq. 6.3-9)}$$

To keep a certain simplicity, this method was also applied for e/D = 1 (Table 6.3-1). For smaller permeability the water level gradient at the structure must be taken into account for the calculation of wave induced forces on pile structures.

e/D [-]	1	2	3	4
P [%]	50	67	75	80
β [-]	1.85	1.5	1.25	1.0

Table 6.3-1: Correction factor β for different relative spacing e/D

270

6.3.3 Floating Debris

In general, the increase of the water level gradient at the groyne structure by trapped but confined floating debris can be neglected. Nevertheless, the drag forces acting on this floating body, will result in additional loads on the groyne piles. The additional horizontal load F_h (kN) induced by a blocking heap of debris is mainly dependent on the distance e between two adjacent piles, the height h_d of the heap and the local surface flow velocity u_s. The given formula does not include the shear stress resulting from the flow underneath the trapped material

$$F_h = 0.5 \cdot \frac{\gamma_w}{g} \cdot e \cdot h_d \cdot u_s^2 \qquad \text{(Eq. 6.3-10)}$$

where

F_h (kN) horizontal force to single pile
h_d (m) thickness of floating debris
u_s (m/s) surface velocity

The lever arm should consider the most unfavourable condition, which is the distance between design bed level and design water level or the crest level of the individual piles. Due to the damping effect of floating material on wave action, the wave structure interaction is not considered in combination with floating debris.

6.3.4 Load Combinations

The piles are stressed by current induced loads together with forces resulting from either waves or floating debris. Waves are absorbed by the floating debris, so that the forces caused by waves and floating debris do not act simultaneously and only the more unfavourable load has to be considered in the pile design.

6.4 PILE EMBEDMENT LENGTH

6.4.1 Modified Blum Method

All loads acting on a vertical groyne pile must be transmitted into the existing subsoil. The horizontal force and the bending moment are compensated by the earth pressure which is limited by the embedment module of the soil. The embedment length is the statically required pile length below the design bed level including the design scour depth (Fig. 6.4-1).

Besides the prevalent loads, the required embedment length t_e of groyne piles is strongly dependent on the soil characteristics. Because the embedment module of the in situ soil is not known, several approximate methods were developed, to calculate the minimum value t_e. A comparatively simple approach is given by Blum (1932, in EAU, 1990, see Fig. 6.4-2). This method was originally developed for the design of single pile and multi-pile dolphin structures under impact loads (e.g. ship manoeuvres) and has therefore certain limitations, for that reason it should only be used for preliminary design purposes.

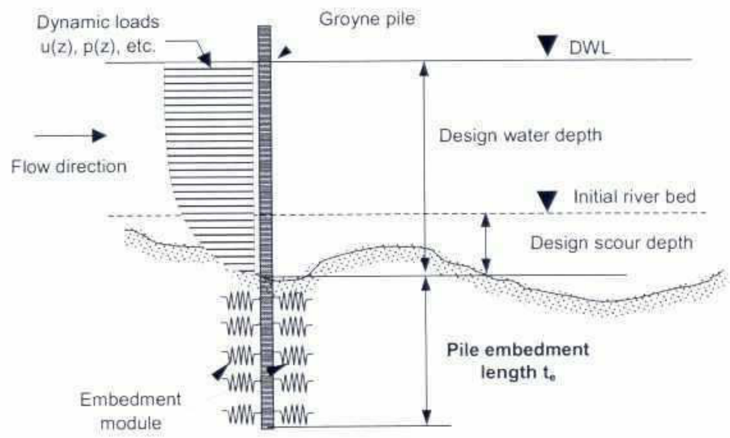

Fig. 6.4-1: Definition of the pile embedment length

A more sophisticated method on basis of a spatial earth pressure distribution, which requires more intense computations, has been applied for the development of the design diagrams given for standardized protection measures of the categories SC2 and SC3 (see Design Plate I). This method is explained briefly in the following.

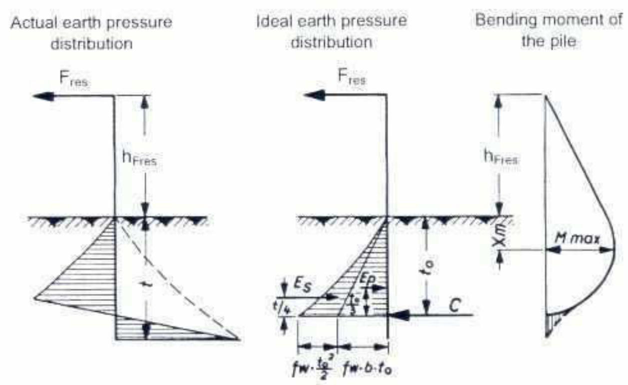

Fig. 6.4-2: Assumptions and simplifications of the Blum method
(adapted from EAU, 1990)

The first step in applying the Blum method is to reduce the actual single and linear loads to a resulting force F_{res} with the lever arm h_{Fres} in relation to the design river bed level. The resulting force is given by

$$F_{res} = \sum_{z=0}^{z=-h} F_h(current) + F_h(wave) + F_h(float.debris) \qquad \text{(Eq. 6.4-1)}$$

whereas the respective resulting moment M_{res} can be derived by considering the specific lever arm $z_{s,i}$ of the different load components:

$$M_{res} = \sum F_h(current)\cdot z_{sc} + F_h(wave)\cdot z_{sw} + F_h(float.debris)\cdot z_{sfd} \qquad \text{(Eq. 6.4-2)}$$

The lever arm h_{Fres} of the resulting force is defined by:

$$h_{Fres} = \frac{M_{res}}{F_{res}} \qquad \text{(Eq. 6.4-3)}$$

The minimum embedment length t_e is calculated following the pre-condition that the pile bending moment is equal to zero. Applying the ideal earth pressure distribution, the fictitious embedment length t_0 at the ideal centre of moments (M = 0, see Fig. 6.4-2) has to be calculated iteratively. Following the definitions given in Fig. 6.4-2 t_0 can be derived from

$$\sum M = 0 \rightarrow F_{res}(h_{Fres} + t_0) - f_w \frac{D\cdot t_0^2}{2}\cdot\frac{t_0}{3} - f_w\frac{t_0^3}{6}\cdot\frac{t_0}{4} = 0 \qquad \text{(Eq. 6.4-4)}$$

$$F_{res}(h_{Fres} + t_0) - f_w\left(\frac{D\cdot t_0^3}{6}\cdot\frac{t_0^4}{24}\right) = 0 \qquad \text{(Eq. 6.4-5)}$$

$$t_0^4 + 4D\cdot t_0^3 - \frac{24}{f_w}\cdot F_{res}\cdot t_0 - \frac{24}{f_w}\cdot F_{res}\cdot h_{Fres} = 0 \qquad \text{(Eq. 6.4-6)}$$

with

D	(m)	pile diameter
f_w	(t/m^3)	coefficient of passive earth pressure $f_w = \gamma \tan^2(45°+\varphi/2)$
γ	(t/m^3)	density of subsoil
t_0	(m)	fictitious embedment length at the ideal centre of moments

The quadratic term of t_0 (second term in (Eq. 6.4-4)) considers the linear increase of earth pressure with depth directly behind the pile (idealized pressure distribution), whereas the cubic term of t_0 (third term in (Eq. 6.4-4)) considers the exponential increase of the spatial extent of the activated earth pressure.

When taking into account a spatial earth pressure component in transversal direction (parallel to groyne axis), it has to be borne in mind that the initiated spatial earth pressure of two adjacent piles will overlap at a certain critical depth t_{crit}, which must be considered in the computations. Below t_{crit} the earth pressure is increasing only linear with increasing penetration depth of the piles, therefore the assumption regarding the earth pressure distribution in Fig. 6.4-2 is not appropriate for narrow spaced groynes. The actual (qualitative) distribution of the passive earth pressure is shown in Fig. 6.4-3.

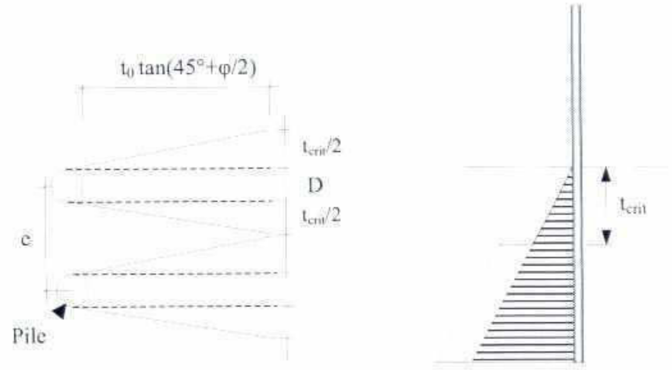

Fig. 6.4-3: Overlapping spatial earth pressures of adjacent piles at critical depth t_{crit}

For a small permeability of the groyne (P = 50 to 60%) the cubic term in (Eq. 6.4-4) can be neglected and instead the diameter D in the quadratic term should be replaced by the distance e between two adjacent piles to derive the minimum theoretical embedment length:

$$\sum M = 0 \rightarrow F_{res}(h_{Fres} + t_0) - f_w \frac{e \cdot t_0^2}{2} \cdot \frac{t_0}{3} = 0 \qquad \text{(Eq. 6.4-7)}$$

The actual minimum embedment length is given by

$$t_e = f \cdot t_0 \qquad \text{(Eq. 6.4-8)}$$

The correction factor f is required to approximate the actual moment distribution and earth pressure condition as described in Fig. 6.4-2 and is f = 1.2 for normal cases. If no bed protection/ falling apron is provided (compare Fig. 6.6-2) this value should be increased to f = 1.5.

To cover inaccuracies in the prevailing boundary conditions the design pile embedment length t_{design} should be determined by the following steps

(1) compute the theoretical minimum required pile embedment length [t_e]
(2) add a safety margin of 25% for consideration of variations in the subsoil
(3) make sure that minimum pile embedment length is always above 5 m

$$t_{design} = 1.25\, t_e \geq 5.0 \text{ m}$$

Because of the rather time consuming calculations, design graphs have been prepared for the easy determination of the required embedment length. The graphs are based on the results of extensive calculations and are shown in Design Plate "I".

6.5 PILE BENDING MOMENT

Groyne piles are almost entirely burdened by horizontal loads. These create a bending moment in the pile, which must be designed appropriately.

The bending moment distribution in the pile is given by

$$M_x = F_{res}\left(h_{Fres} + x\right) - f_w\left(\frac{D \cdot x^3}{6} + \frac{x^4}{24}\right) \qquad \text{(Eq. 6.5-1)}$$

The location x_m of the maximum bending moment below the design river bed has to be derived iteratively

$$\frac{x_m^3}{6} + \frac{D \cdot x_m^2}{2} = \frac{F_{res}}{f_w} \qquad \text{(Eq. 6.5-2)}$$

or

$$F_{res} = \frac{f_w}{6} \cdot x_m^2\left(x_m + 3 \cdot D\right) \qquad \text{(Eq. 6.5-3)}$$

so that

$$M_{max} = \frac{f_w}{24} \cdot x_m^2\left[3x_m^2 + x_m\left(4 \cdot h_{Fres} + 8 \cdot D\right) + 12 \cdot h_{Fres} \cdot D\right] \qquad \text{(Eq. 6.5-4)}$$

The evaluation of these formulae has also been included into the design graphs, presented in Design Plate "I". One graph each has been prepared for steel piles and pre-cast concrete piles, showing their main required dimensions.

6.6 BED PROTECTION AND FALLING APRON

The local erosion of the riverbed near a permeable groyne head can be prevented by a bed protection or a falling apron. A bed protection consists of a cover layer placed on a filter mat, whereas the falling apron material is dumped directly on the riverbed. Since under the prevailing flow conditions the placing of the filter mat is rather complicated, falling aprons are the only practicable solution at present (Fig. 6.6-1). The dimensioning of the falling apron units (preferably CC-blocks) and the respective volume required can be derived from the formulae given in Chapter 5.

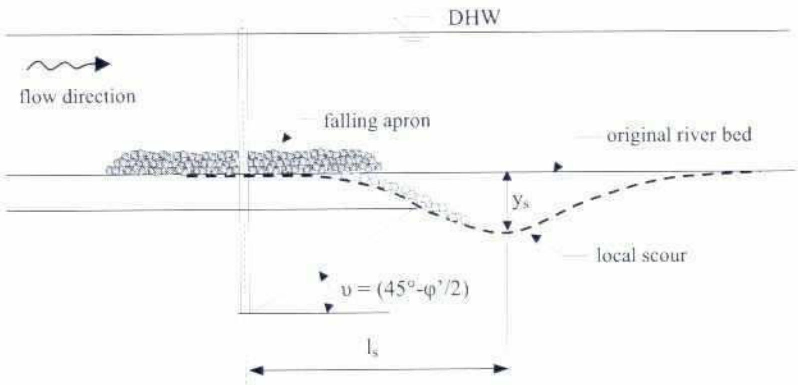

Fig. 6.6-1: Expected local scour near a groyne head with falling apron

However, if possible, a solution without any bed protection or falling apron should be considered for permeable groynes (Fig. 6.6-2), because - although it would increase the required embedment length of the piles - it is in general more economic as compared to additional protective measures at the river bed.

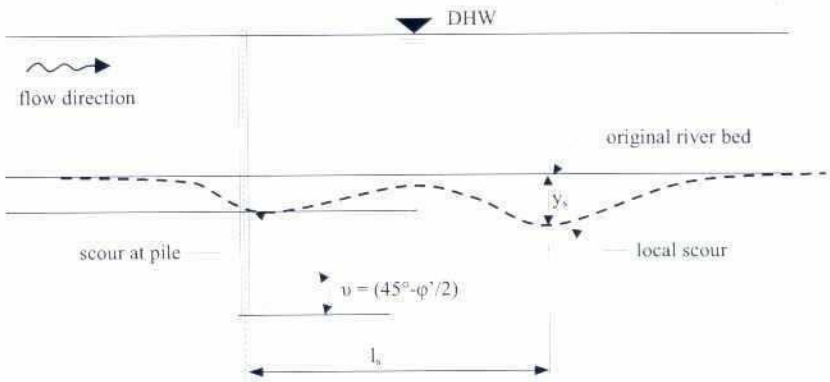

Fig. 6.6-2: Design scour profile – without scour protection on river bed

207

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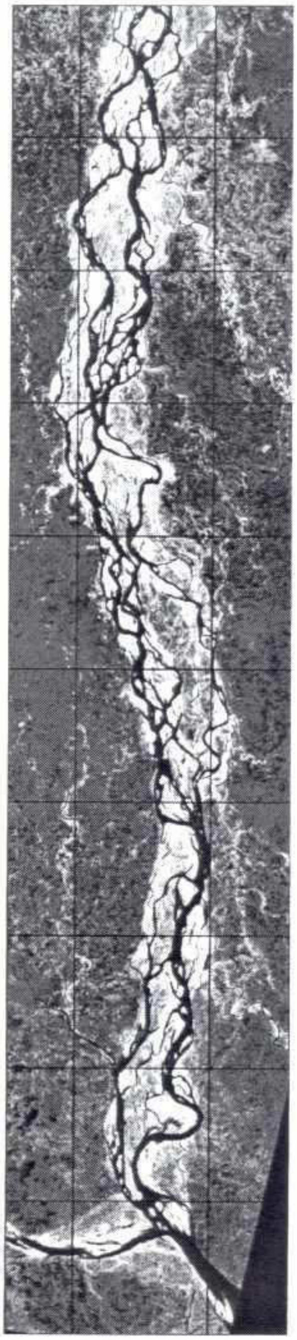

PART 2

MANUAL

PART 2 - MANUAL

Contents

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Figures

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Tables

7.1 EARTH WORKS

7.1.1 General Description

7.1.1.1 Excavation in Dry Condition

For the situation of remote site locations within Bangladesh a labour intensive method for earth moving is still the preferred method. In order to estimate the workforce required for excavating, transporting and placing/filling of soil the diagram presented in Figure 7.1-1 may serve as a general guidance.

Exemplary, the following can be obtained from the said diagram for a given situation:

- Transport distance between excavation (e.g. borrow pit) and place of fill: 175 m
- Height difference between bottom of borrow pit and top of embankment: 4.50 m
- Resultant manpower for 100 m^3 of soil: 80 man-days

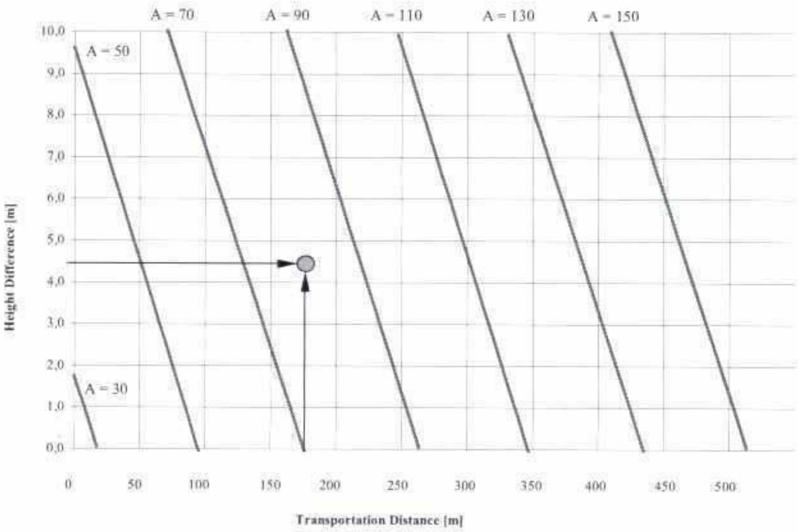


Figure 7.1-1: Diagram for determination of manpower required for manual excavation, transport and filling of 100 m^3 of soil

Mechanical excavation, transportation and filling can accelerate the work progress, but the decision should be made in consideration of the volume of work to be accomplished within a given construction window, as well as with due regard to the particular local and social situation around a specific construction site.

As a rough guideline the productivity of mechanical equipment for excavation works may be taken from Table 7.1-1 and Table 7.1-2. Therein, the soil classification shall be an indicator for:

"Light/easy" : soft, weak soil that can very easily be cut and excavated (relative high productivity);
"Medium" : normal soil, i.e. in a compacted/medium dense deposited stage, and
"Heavy/difficult": densely deposited soil or stiff cohesive soil that can not be easily cut and excavated.

Soil	Dragline Bucket			Clamshell Bucket		
Classification	V = 0.5 m³	V = 0.9 m³	V = 1.5 m³	V = 0.5 m³	V = 0.9 m³	V = 1.5 m³
Light/easy	32	52	80	25	40	65
Medium	20	34	52	15	25	40
Heavy/difficult	14	24	38	10	18	30
Under water	About 60 % of above values			About 60 % of above values		

Table 7.1-1: Excavation capacity for different bucket types (m^3/hr)
(V = net bucket volume, top flat)

Transfer and/or distribution of excavated / cut soil may be carried out by bulldozer, but the economical transfer distance us much below 100 m, commonly between 30 and 60 m only.

Bulldozer	Shield	Normal (Medium) Soil			Heavy (difficult) Soil		
Capacity	Width	D = 20 m	D = 60 m	D = 100 m	D = 20 m	D = 60 m	D = 100 m
45 HP	2.20 m	80	30	20	(unsuitable)		
100 HP	3.35 m	165	75	50	70	35	25
150 HP	3.90 m	250	110	80	100	50	35

Table 7.1-2: Bulldozer transport capacities (m^3/hr)
(D = distance of transfer)

It has to be recognised that the actual productivity depends on several important factors and parameters, including boom length, turning radius and angle of the excavator, equipment availability factor (preventive maintenance is a must!), etc.

7.1.1.2 Excavation Equipment (Wet Condition)

"Wet Excavation" within this context shall mean excavation below water, which is being carried out by shore-based mechanical equipment. Shore-based crawler cranes with clamshell buckets are ideal for such works. However, the productivity and quality of work depends largely on the professionalism of the crane operator. Productivity estimates may be based on the figures presented in Table 7.1-1, but due consideration must be given to the loss of hourly productivity to about only 60 % as compared to dry excavation with the same equipment.

7.1.1.3 Compaction Equipment

The importance and respective requirements for the compaction of any fill for embankments, flood protection dykes, etc. is obvious. Highly sophisticated compaction methods and equipment may not be considered for the regular type of standardized bank protection and erosion prevention measures.

The most difficult fact to be considered when planning and deciding on the compaction method and equipment is the fact that the soil to be compacted cannot be selected to comply with a criterion "suitable for compaction". Rather the soil available in the immediate vicinity of a construction site, including adjacent chars, has to be used in the works. It is for that reason that ordinary compaction methods can be considered appropriate under the circumstances and in consideration of what is commonly available from equipment resources. The common methods include the use of

- Bulldozers passing the filled surface area x-times cross-wise;
- Tamping plate (drop weight with a certain surface area, handled by a crawler crane);
- Heavy-duty internal combustion tamper;
- Surface vibrator, also named plate vibrator;
- Roller compactor, pneumatic roller compactor;
- Tamper roller (sheep-foot roller)

In any case the effectiveness of the compaction equipment has to be verified by field tests. Thereby the soil characteristics, the thickness of individual fill layers to be compacted and the number of passes to achieve the specified degree of compaction must be determined. Table 7.1-3 may be used as a guideline only for determining the equipment resources and capacity.

Method of Compaction / Type of Equipment	Suitable for Compaction of:	Capacity / Weight	Standard Layer Thickness	Number of Passes	Compaction Volume per Equipment-Hour
Tamping plate / crawler crane	Non-cohesive soil	2 - 3 t	50 - 70 cm	3 – 4	25 - 50 m³
Heavy-duty internal combustion tamper	Cohesive / non-cohesive soil	0.5 t	30 - 40 cm	3 – 4	15 – 20 m³
		1.0 t	40 - 50 cm	3 – 4	25 – 30 m³
Surface (plate) vibrator	Non- to slightly cohesive soil	0.5 - 0.7 t	40 - 50 cm	3 – 4	24 - 30 m³
		> 1.7 t	50 - 70 cm	3 – 4	40 - 50 m³
Vibro-roller (self-driven)	Non- to slightly cohesive soil	1.2 - 2.3 t	30 cm	4 – 6	35 - 50 m³
		3 - 6 t	40 - 50 cm	4 – 5	90 - 110 m³
Pneumatic roller (self-driven)	Cohesive / non-cohesive soil	9 - 12 t	15 - 20 cm	8 - 12	65 - 100 m³
Tamper roller (sheeps-foot roller)	Cohesive soil, free of stones	3.5 t	18 - 20 cm	8 - 12	32 - 48 m³
		6 t	18 - 20 cm	8 - 12	65 - 95 m³

Table 7.1-3: Typical capacity of compaction equipment
(typical for the soil usually encountered along the rivers of Bangladesh)

247

7.1.2 Sample Specifications

7.1.2.1 Definition

Earthworks are

- "common excavation" as defined under Section 7.1.2.3, and
- "filling works" as defined under Section 7.1.2.4.

Earthwork materials are defined to apply to the Specifications and the Works as follows:

- "top soil" is the top layer of soil containing a higher proportion of organic material;
- "suitable material" comprises all material obtained from excavations within the Site or from borrow pits and which is approved by the Employer as acceptable for use in the Works;
- "unsuitable material" is any other than suitable material and shall comprise:
  - material from swamps, marches and bogs or other material with a high organic content;
  - peat, logs, stumps and perishable materials;
  - material susceptible to spontaneous combustion;
  - clay of liquid limit exceeding 90% and/or plasticity index exceeding 65%.
- "soft" material shall mean all material, whether suitable or unsuitable, but other than that defined as rock hereunder.
- "rock" is any hard natural or artificial material requiring the use of blasting or approved pneumatic tools for its removal.

7.1.2.2 Clearing of Construction Site

The areas to be used for the Works shall be cleared by removing vegetation, trees, any obstacle, debris, rubbish, foundations etc. or otherwise unsuitable matter, which would prevent the technically sound execution and completion of the Works.

Stumps and major roots shall be grubbed out and all combustible material arising shall be gathered into windrows and burnt. The Contractor shall take precautions to prevent the spread of fire to adjacent land.

Notwithstanding the provisions of this Subsection, the Contractor shall preserve established grass cover on existing embankments that fall within the Site, but which have been designated on the Drawings or as may be directed by the Employer for retention in their present state.

7.1.2.3 Common Excavation

(a) Definition

The term "common excavation" under this Subsection shall mean stripping, excavation or removal of any type of material on or near bank for construction pits or embankment revetment and bed protections, whether in dry or in wet condition, to the lines and grades shown on the Drawings or as required by the Employer.

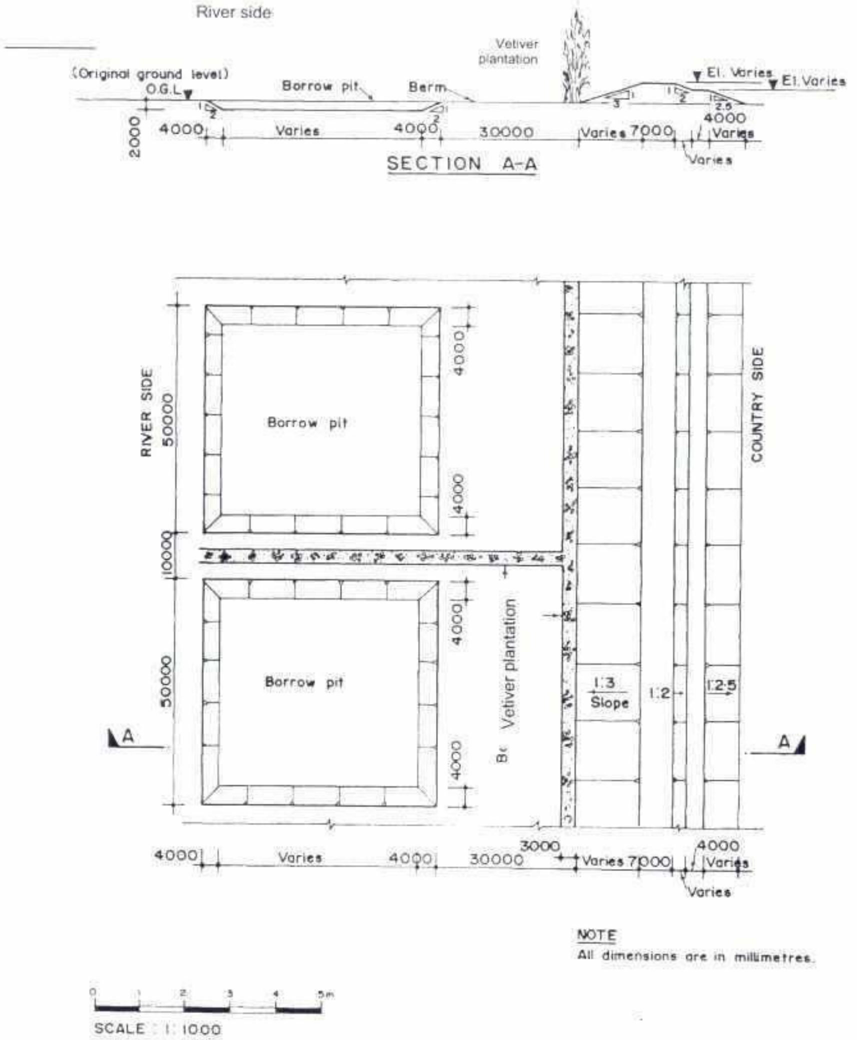


Figure 7.1-2: Recommended arrangement of borrow pits along embankments

(b) Method Statement

Notwithstanding the requirement to submit a detailed work method statement as part of the Tender, the Contractor has to substantiate the method statement prior to commencement of the common excavation. The Contractor shall within 28 days of the Letter of Acceptance submit in writing to the Employer for approval his final proposal for carrying out the respective work.

The proposal shall cover method and sequence of working, equipment to be employed, and shall include details of the methods for controlling adherence to the design level and slopes. Common excavation shall not commence until the Employer approves the proposals and the control methods intended to be used.

(c) Top Soil Stripping

Top soil shall be removed to the lines designated in the Drawings or as directed by the Employer and shall be deposited in separate heaps for re-use, free of weeds.

(d) Excavation

At all times during construction, the Contractor shall adopt excavation procedures, which ensure that the stability of any slope will not be impaired. The Employer's approval of excavation procedures shall in no way relieve the Contractor of his responsibility for safeguarding the stability of all slopes excavated under this Contract.

Excavation shall be carried out to the lines, levels and profiles shown on the Drawings or to such other lines, levels and profiles as the Employer may direct or approve in writing.

The Contractor shall carry out any excavation in a way to minimize disturbance to the surrounding ground. Particular care shall be taken to maintain stability when excavating in close proximity to existing works.

Excavated material shall be disposed of or may be incorporated into the Works either directly or after stockpiling, as stipulated in the Bill of Quantities or otherwise directed by the Employer. Only those materials meeting the appropriate specifications may be incorporated into the Permanent Works.

The Contractor shall remove unsuitable material from the Site as ordered or agreed by the Employer.

With the exception of reasons falling under force majeure the Contractor is responsible for keeping all excavations free from water from whatever cause arising. Any surface or subsurface water flow, entering into or adjacent to the excavation, shall be satisfactorily controlled and the Contractor shall provide such pumping capacity and other measures as are required, all to the satisfaction of the Employer.

Where required, the Contractor shall properly support the sides of excavations and shall be responsible for their safety.

The Contractor shall notify the Employer of any unusual soil encountered during excavation, without delay.

(e) Tolerances

Excavated levels shall be within a tolerance of ± 100 mm. Slopes of embankments and cuttings shall be accurate within +100 mm/-000 mm within 10 m measuring distance.

If for any reason whatsoever excavations were carried out by the Contractor beyond permissible tolerances to their true lines and levels as shown on the Drawings or differently from the direction of the Employer, the Contractor shall, at his own cost, make them good to the required lines and levels with compacted sand or other approved material, and in such a manner as the Employer may direct.

(f) Approval of Excavation

After any excavation is completed accurately to the lines and levels required for the Works, the Contractor shall notify the Employer, who may carry out an inspection.

If, after such inspection, the Employer requires additional excavation to be carried out, the Contractor shall do so to such profiles, dimensions or levels as the Employer may direct.

The Contractor shall commence any subsequent work only after the Employer has approved the excavation depths and slopes, including the compaction of the excavated surfaces, where required.

The Contractor shall maintain open excavations in an approved condition. Where required, he shall rectify the effects of deterioration due to surface water or any other act.

7.1.2.4 Filling Works

(a) Definition

Filling works comprise of filling and compaction of any land, construction of embankments or groynes, filling and compacting of construction pits and excavations, back filling of structures, etc., to designed levels.

(b) Method Statement

Notwithstanding the requirement to submit a detailed work method statement along with the Tender, the Contractor has to substantiate the method statement prior to commencement of filling works. The Contractor shall within 28 days of the letter of Acceptance, submit in writing to the Employer for approval his final proposal for carrying out the respective work.

The proposal shall cover method and sequence of filling and compaction and shall include details of the compaction plant as well as methods for adjusting the moisture content of the different fill materials.
Filling shall not commence until the Employer approves the proposals and the materials intended to be used.

(c) Fill Material

Fill and backfill material other than dredged or excavated material obtained at Site or from borrow pits within the Site shall be sand with a maximum silt content (grain size less than 0.063 mm) of 5% for material with steep grain size distribution curves, or of 10% for material with flat curves.

Materials selected by the Contractor for the individual purposes, have to be tested by him and representative samples shall be submitted to the Employer along with test reports for approval before starting the work. Only approved materials shall be used. The samples will be detained by the Employer and used for comparison.

Suitable material for construction of embankments shall be obtained from common excavation or dredging, respectively from designated borrows pits adjacent to the Site. It shall be sand with silt content of not more than 15 to 20%. It shall be free of all organic and otherwise undesirable matter and be approved by the Employer.

The top layer of any unsuitable material covering the borrow pits shall be removed to a depth where suitable material can be obtained. Unsuitable material shall be interim stored and refilled to the borrow pits after completion of the Works, or otherwise being utilized as directed by the Employer.

(d) Filling and Backfilling

Prior to placing suitable fill material on any area, the foundation area shall be further excavated, shaped, drained and cleaned of any weak or other objectionable material, all as per requirements of the Employer and in consideration of Section 7.1.2.4(e).

The Contractor shall not commence filling of any area or backfilling of any structure until the Employer has given his respective approval.

The Contractor shall take representative samples of the fill material during placement as directed by the Employer. At least three samples shall be taken per day of filling works, and a particle size determination be made. Any material found not in conformity with the Specifications or the otherwise directions of the Employer will be rejected and shall be removed from Site by the Contractor. Rejection may be made at the source, on the transporting vehicle, or in place. Acceptance of fill will be made only after the materials have been dumped, spread and compacted in place.

Fill shall be carefully placed to avoid excessive loads on any temporary or permanent structure or part thereof.

(e) Compaction

The type of compaction equipment employed by the Contractor shall be such that the required densities can be produced consistently. The foreseen equipment requires the approval of the Employer.

The Contractor shall perform trial compactions for each type of fill material, in order to establish the appropriate method to achieve optimum compaction. Based on the results of such trial compactions the optimal water content during compaction, the necessary number of compacting cycles and the thickness of built-in layers shall be determined in consideration of the type and weight of Contractor's compaction machines, all subject to the Employer's approval.

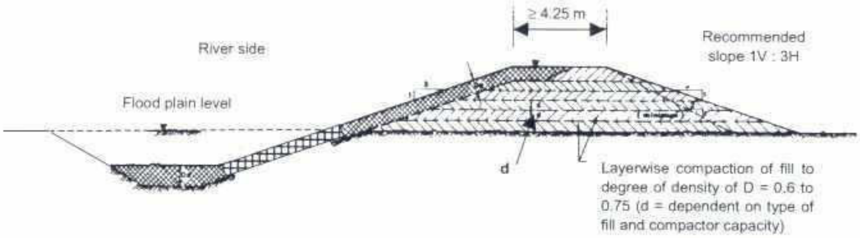


Figure 7.1-3: Recommended build-up of embankment

Before starting any filling works, the existing degree of density of the natural ground has to be determined by the Contractor to the Employer's requirements. The Employer will thereupon decide whether compaction of the existing soil is necessary. The required density values shall meet the requirements as indicated below and shall be confirmed to a depth of 0.5 m.

Fill materials shall be placed in layers uniformly spread, moistened as far as required and then compacted. The thickness of the layers depends on the material, type of vibrators and results of compaction tests executed at Site, taking into account an appropriate safety margin.

The Contractor shall verify the achieved degree of density, as the filling and compaction works proceeds. The degree of density of each layer of fill shall be checked as per DIN 18126 through taking and testing undisturbed soil samples. For each 500 m^2 of compacted surface three adjacent points are to be investigated by density tests.

All samples are to be taken and tests performed in the presence of the Employer. The Contractor shall submit results of all such tests on approved form. The costs for sampling and testing shall be borne by the Contractor and are deemed to be covered by surcharges included in the relevant unit rates.

The degree of density of each layer of fill shall be checked as per DIN 18126 through taking and testing undisturbed soil samples.

$$D = \frac{\max n - n}{\max n - \min n}$$

with,

max n	=	Degree of porosity of soil in loosest state, in the dry
min n	=	Degree of porosity of soil in densest state
n	=	Degree of porosity of compacted soil.

For each 500 m^2 of compacted surface three adjacent points are to be investigated by density tests.

All samples are to be taken and tests performed in the presence of the Employer. The Contractor shall submit results of all such tests on approved form. The costs for sampling and testing shall be borne by the Contractor and are deemed to be covered by surcharges included in the relevant unit rates.

The required degree of density is as follows:

Layer	Density D (DIN 18126)
Up to 0.5 m below present surface	0.35 - 0.45
Fill for embankments, dams	0.6 - 0.75

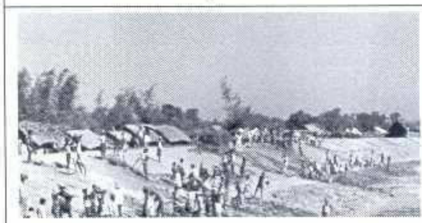
Individual test results may be down by 5%, but the mean value of adjacent test points must at least correspond to the specified values. Should two testing methods lead to different judgment, the more unfavorable result will be considered only.

If tested densities should fall below the above limits at any place, recompaction of such area shall be carried out by the Contractor and new tests executed thereafter.

Portions of fill which, in the opinion of the Employer, cannot be adequately compacted with rollers or surface vibrators due to inaccessibility shall be placed in layers not exceeding 150 mm and compacted to the specified density by means of approved power tampers, all at no additional costs to the Employer.

Excavation by manual labour

Borrow pit excavation



Slope profiling by labour

Compaction control (sand replacement method)

Figure 7.1-4: Earthworks by human labour


Dozer and truck operation


Common mechanical excavation


Dragline excavation and slope profiling


Dragline and truck operation

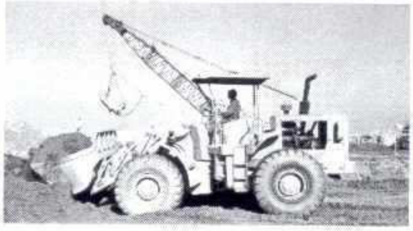
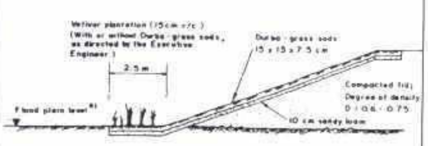

Dozer for earth moving

Figure 7.1-5: Earthworks with heavy equipment

7.2 TURFING OF EMBANKMENT SLOPES

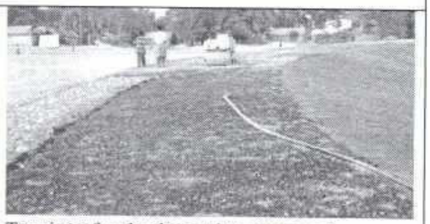
Slopes and areas of embankments, which are not exposed to current and wave action, can be provided with turf, for which grass of suitable quality such as Durba and Vetiver should be used to receive a dense growth of grass turf within the shortest period of time. The slopes and areas should be free of unsuitable soil and material such as roots, stumps etc. and tamped properly to compact any loose soil.

Placing and securing GeoJute-SoilSaver, laying of Durba grass sods



Typical cross-section of embankment with bio-technical revetment (Durba grass sods placed on GeoJute-SoilSaver, Vetiver as toe protection)

Rain cuts in the unprotected slope prove the need for protection also on the lee-side of the embankment



Topping of embankment by compacted sand-khoa layer

Figure 7.2-1: Turfing works

Turfing can be achieved either by grass sods or by seeding turf on the embankment. Grass seeds shall be spread at a rate of 10 g/m² neatly on a 10 cm thick layer of fertile top soil and secured appropriately against surface erosion e.g. by a cover of GeoJute-Soil saver netting, which is to be placed with overlaps between adjoining nettings of at least 15 cm. The nettings are to be pegged down by staples at about 40 cm intervals, which should be reduced to 15 cm along overlaps and edges. The top and bottom ends of the nettings shall be secured by digging the ends about 30 cm deep into the soil.

A second grass seeding at 10 g/m² is to be spread after the GeoJute-Soil Saver is in place and the surface tamped finally, the nettings flush with the soil surface. The seeding needs proper irrigation for a certain period to obtain the required result and simple fencing to prevent destruction by pedestrians and animals shall protect the respective areas. When the grass has grown, regular cut and maintenance is required.

Turfing with grass plantation should be done in the proximity of the embankment. It requires an area equal to the surface area of the slopes of the embankment where turfing will be done. Sods (15 cm x 15 cm x 7.6 cm) of the fully developed turf are cut and placed closely on the embankment. If the turf turns to yellow, water as well as fertiliser should be added.

7.3 GRANULAR FILTER LAYERS

7.3.1 General Description

Granular filters can be graded sand and gravel filters or mixed filters. The former can only be placed above the water level. Placing shall start immediately after trimming of the embankment slopes at its toe in horizontal layers and in such a manner that washing out and disintegration of filter material are avoided. Prior to laying the subsequent layer, the achieved slope and layer thickness of the initial layer must be confirmed through measurements.

The method of placing mixed granular filters below the water level is mainly governed by the flow situation prevailing at the time of carrying out the work. For placing in-situ the following methods are possible:

- Using large diameter tremie-pipes to release the graded filter material directly at the bank slope or river bottom;
- Utilizing large-size buckets, which can be lowered by crane and opened directly above the river bed and by gently swinging the cranes' boom producing a filter carpet, and
- Using large clamshell, which is lowered to the river bottom or bank slope to release the mixed filter material directly at the selected spot.

The mixed granular filter material can also be filled in jute bags. After sealing the bags they can be dumped at a controlled pattern to achieve at least 2 layers of bags within the defined area

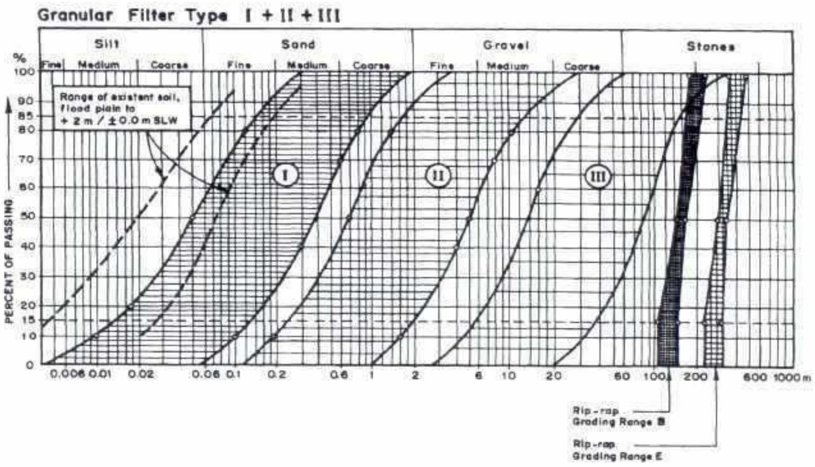


Figure 7.3-1: Typical granular filter for sandy silt strata

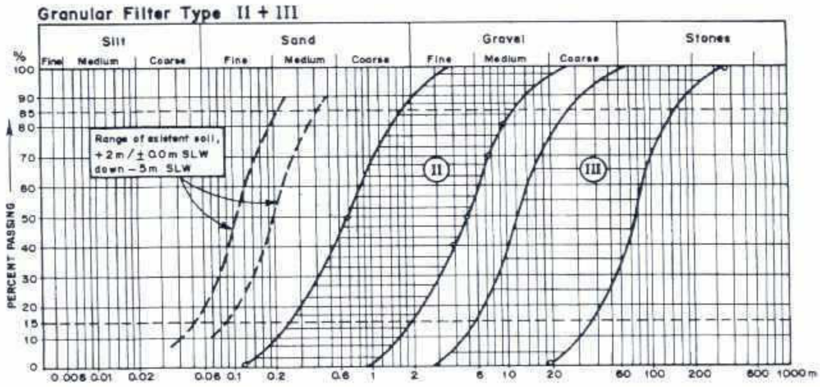


Figure 7.3-2: Typical granular filter for sandy soils

Figure 7.3-3: Placing of multi-layer granular filter and rip-rap cover layer

7.3.2 Material Specification for Granular Filters

7.3.2.1 General

Granular filters comprise at least of a two-layer system

- sand filter (filter stage 1), and
- gravel filter (filter stage 2).

These filter systems are suitable for placing in dry condition. Each filter shall have an in-situ layer thickness of at least 30 cm.

Where the granular filter cannot comprise adequately large grain size as blocker against the designed cover layer a suitable graded cushion layer (also referred to as filter stage 3) or an additional sublayer must be provided in coordination with the Employer.

For application below water level a one-layer granular filter may be applied, the grain size range of which must be a suitably mixed gradation corresponding to a two-layer system. Such mixed granular filters (sand-gravel filter) shall have an in-situ thickness of at least 60 cm.

7.3.2.2 Sand Filter (Filter Type I)

Sand shall be granular, cohesionless and free from organic impurities. The grain size distribution will generally be in the range of $D_{15} = 0.02$ mm to $D_{60} = 0.4$ mm, but must be suitably adjusted to the grading of subsoil to be protected. The coefficient of uniformity shall be in the range of 5-8.

Filter sand shall be obtained by the Contractor only from reliable sources (e.g. within Bogra District, Dinajpur District or Sylhet District), and shall be sieved and suitably blended to the requirements and subject to Employer's approval.

7.3.2.3 Gravel Filter (Filter Type II)

Gravel filter material shall mainly be granular, cohesionless and free from organic impurities. The grain size distribution may generally be in the range of $D_{15} = 0.3$ mm to $D_{60} = 5.0$ mm, but must be suitably adjusted to the grading of sand filter and cushion layer. The coefficient of uniformity shall be in the range of 5-8.

The components of a gravel filter shall be carefully selected and sieved from crushed stone or gravel being free from dirt and any other objectionable matter.

7.3.2.4 Cushion Layer (Filter Type III)

The cushion layer shall be of granular/crushed material. The gain size distribution may generally in the range of $D_{15} = 6.0$ mm to $D_{60} = 100$ mm, but must be suitably adjusted to grading range of the gravel filter and the cover layer. The coefficient of uniformity shall be in the range of 5-8.

The components of a cushion layer shall be carefully selected and sieved from crushed stone or gravel being free from dirt and any other objectionable matter.

7.3.2.5 Khoa Filter

Khoa filter material shall be produced from first class bricks or picked Jhama khoa only. The grain size components must be carefully produced, selected and blended to suit the required grading of Sections 7.3.2.2, 7.3.2.3 and/or Section 7.3.2.4.

7.3.2.6 Mixed Granular Filters (Filter Type IV)

Mixed granular filter material shall mainly be granular, cohesionless and free from organic impurities. The grain size distribution will be generally in the range of D_{15} = 0.4 mm and D_{60} = 100 mm, but must be suitably adjusted to the grading of the subsoil to be protected respectively to the designed gradation of the cover layer.

The components shall be carefully selected and sieved from sand and crushed stone, natural gravel or khoa, being free from dirt or any other objectionable matter. Sections 7.3.2.2, 7.3.2.3 and 7.3.2.4 apply analogously.

7.4 GEOTEXTILE FILTER MATERIALS AND INSTALLATION

7.4.1 General Description

Geotextiles shall be placed on the slope of an embankment in strict compliance with the manufacturer's instructions. This holds in particular for the method of jointing and overlapping. Overlaps on the slope shall be arranged down the slope. If an overlap transverse to the slope is unavoidable, then the upper geotextile should be laid on top of the lower one. Overlaps shall be at least 50 cm with additional stitching when placed in the dry and they should be staggered by minimum 150 cm. When placed under water, the overlapping should be minimum 1.0 m for water depths up to 4.0 m, minimum 1.5 m for water depths up to 8.0 m and minimum 2.0 m for greater water depths.

The geotextile filter material shall not be torn, punctured or damaged otherwise. When laying the geotextile, the surface of the embankment slope should be a relatively smooth plane, free of obstructions, depressions and soft pockets of material. Depressions must be filled with compacted material; otherwise the fabric could bridge and tear when the cover layer is placed. Free ends of the laid geotextile shall only be secured by ballasting with stones, bricks or sand bags, but under no circumstances be nailed to the subsoil.

The cover layer should be placed on the geotextile as soon as possible. The material of the cover layer is to be deposited starting from the bottom of the slope.

7.4.2 Material Specifications for Geotextile Filters

Geotextiles, whether geotextile fabrics or tailored geotextile fabrics shall be the make of a reputable manufacturer, and the manufacturer has to produce test certificates of a recognized testing institute to confirm compliance with the specified requirements.

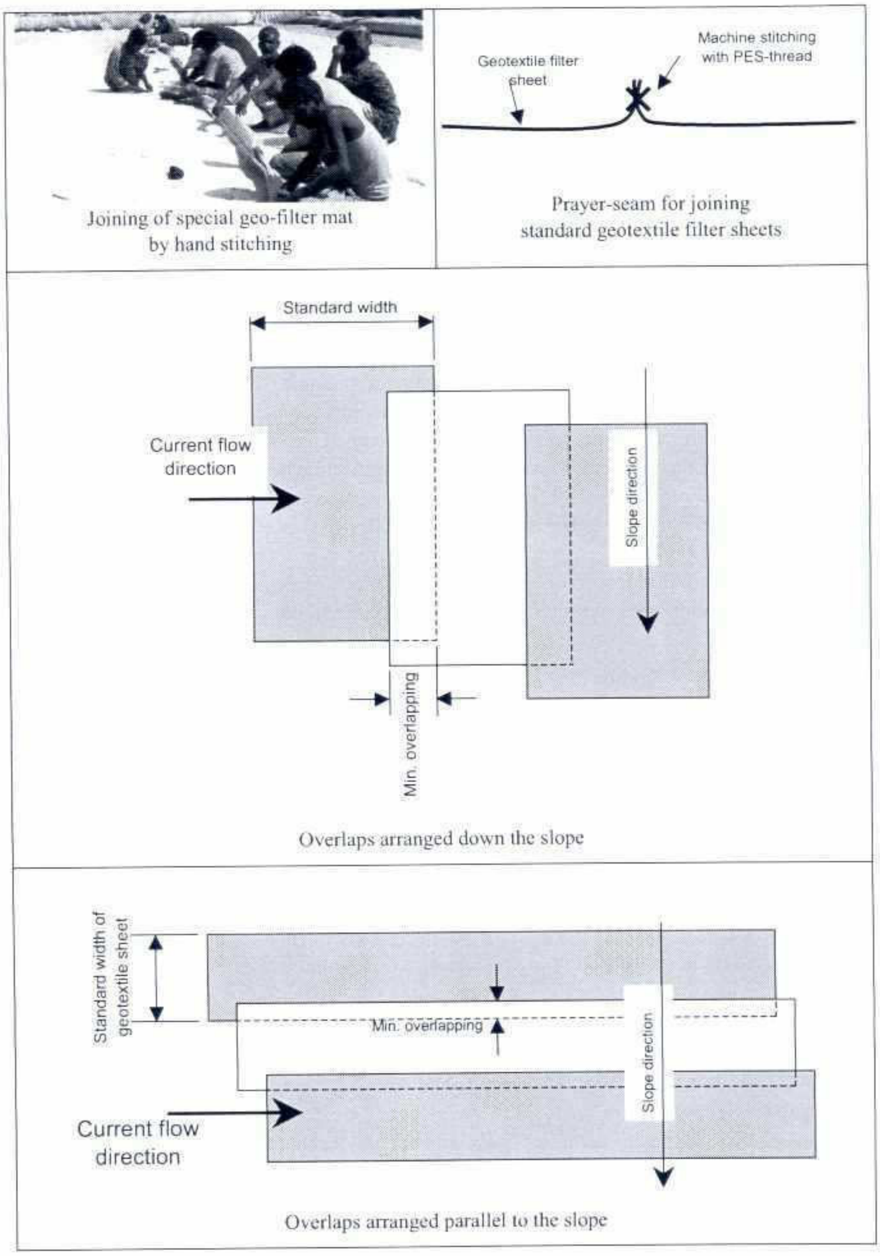
7.4.2.1 Mechanical Properties of Geotextiles

Geo-textile mats may be exposed to considerable tensile loads, which have to be resisted by the material. A minimum tensile strength of 12 kN/m should be guaranteed (new material), which should not decrease below a residual value of 9 kN/m (due to abrasion or chemical/ biological processes in the subsoil or groundwater), to accommodate deformations of the subsoil or hen used as a sub-layer for the launching apron. The risk of local ruptures during construction is mainly governed by

- type and condition of the subsoil;
- shape and weight of dumped material;
- dumping height, in the dry or under water (construction technique),

and other conditions. The required minimum resistance to rupture is given in relation to the weight of armour units. A minimum of 600 Nm is recommended for stones of 30 kg dumped from 2 m height. For heavier units of 60 kg, 1200 Nm are required as minimum.

282



Joining of special geo-filter mat by hand stitching

Prayer-seam for joining standard geotextile filter sheets

Overlaps arranged down the slope

Overlaps arranged parallel to the slope

Figure 7.4-1: Details of geotextile filter installation

7.4.2.2 Geotextile used as Revetment Filter Layer (above berm)

Protective layers of uniformly sized concrete blocks (placed in parallel rows) of more than 50 kg single weight, or stone mattresses of 25 cm to 40 cm thickness require filter fabrics (non-woven needle-punched geo-textile) of the following specifications:

- Material preferably 100 % Polyester (PES), or 50 % Polypropylene (PP) and 50 % PES
- Effective opening size O_{90}: 0.07 mm
- Thickness ≥ 2.0 mm
- Area mass ≥ 350 g/m²
- Min. tensile strength
  - longitudinal > 1200 N/10 cm
  - transversal > 1400 N/10 cm
- Min. elongation
  - longitudinal ≥ 80 %
  - transversal ≥ 50 %
- Puncture resistance > 600 Nm
- Mechanical filtering capacity (soil Type 4)
  - max. quantity of soil passage: 300 grams
  - max. quantity of soil passage in the last test phase: 30 grams
- Coefficient of permeability at a load of 2 kN/m². ≥ 1·10-7 m/s

7.4.2.3 Geotextile used as Sub-layer for the Launching Apron

For the use of geo-textiles underneath a launching apron (articulating mattress) the material must be able to resist potential larger tensile stresses. The fabric (non-woven needle-punched) should have the following specification:

- Material: Polyester (PES)
- Effective opening size O_{90}: 0.07 mm
- Thickness ≥ 5 mm
- Area mass ≥ 700 g/m²
- Min. tensile strength
  - longitudinal > 3200 N/10 cm
  - transversal > 4700 N/10 cm
- Min. elongation
  - longitudinal ≥ 60 %
  - transversal ≥ 50 %
- Puncture resistance > 1200 Nm

286

- Mechanical filtering capacity (soil Types 1 to 3)
  - max. quantity of soil passage: 25 grams
  - max. quantity of soil passage in the last test phase: 2.5 grams
- Coefficient of permeability at a load of 2 kN/m²: $\geq 1 \cdot 10\text{-}7$ m/s

7.4.2.4 Transportation, Storage and Handling of Geotextile

The Contractor shall transport, handle and store all geotextiles in full and in accordance with the manufacturer's instructions. Geotextiles shall be kept wrapped in black PE sheeting to prevent UV exposure until immediately before use in the Works. If the wrapping is damaged during handling it shall be repaired immediately by the Contractor using additional black PE sheeting. Unused portions shall be re-wrapped promptly.

Geotextile fabrics arriving at Site shall be unpacked and stored under covers, well sheltered from direct sunlight, until required for use in the Works. Sufficient ventilation under the shelter shall be provided so as to minimize the effects of high temperature thermo-oxidation.

Geotextiles damaged due to handling or during installation in the Works are to be repaired immediately by the Contractor in accordance with the manufacturers' instructions and to Employer's satisfaction, all at the Contractor's expense.

Delivery notes attached to the material shall be handed over to the Employer.

7.4.2.5 Sewing and Tailoring of Geotextiles

To limit the number of normal over-lapping between individual geotextile filter mats at least two (for under water application) or three (for application in the above water area) are to be joined to larger filter mats. The quantity of geotextile filter material should be calculated on such basis.

The geotextile filter materials should be delivered in factory standard widths of about 5 m to 5.3 m. Joining of geotextile filter mats should be carried out at Site, the methods are subject to the approval of the Employer.

Electric powered, hand-held sewing machines, e.g. Type UNION Special 2200 AS should be provided for this purpose by the Contractor.

Type of seam shall be "Prayer Seam" (see Figure 7.4-1), for heavy-duty application also "Butterfly Seam".

Thread shall be PES white and PES black, which should be supplied from the manufacturer of the respective geotextile material.

Type of stitching shall be only double thread chain stitch. The length of stitch is to be coordinated with the thickness of the basic material. About 2 m to 3 m each of white and black PES-thread are required for 1 linear meter of double thread chain stitch, dependent on the thickness of the geotextile filter material.

For field sewing one team should comprise of three persons. The first one to neatly put together the two edges to be sewn, the second one holding and operating the sewing machine, while the third holds the material tight behind the sewer.

Larger areas can be sewn together by cutting the geotextile filter material to the required length, unrolling each, one above the other and sewing the edges together. Subsequently the mat can be unfolded to its full width.

Seam strength should be 90% of the strength of the geotextile filter material, which has to be confirmed by the Contractor through tests at a recognised material testing institute.

7.4.2.6 Placing of Geotextiles

The geotextiles shall be stored, handled and placed on the embankment slope in strict compliance with manufacturer's instructions, or Employer's instructions.

The black PE-sheeting covering the rolls shall only be removed immediately prior to laying. If the covering is damaged during handling, the Contractor shall repair the covering at his own cost.

The material shall not be torn or punctured or damaged otherwise. If during the course of work damage to the geotextile filter mats is encountered, remedial work shall be carried out as directed by the Employer.

Free ends of laid geotextiles shall only be secured by ballasting with stones or sand bags, but under no circumstances be nailed to the subsoil.

The method of joining and overlapping shall generally be as per manufacturer's instructions and as approved by the Employer. Overlaps on the slope shall be arranged down the slope but against the current flow. If an overlap is required transverse to the slope, then the lower geotextile shall be laid on top of the upper geotextile.

Overlaps shall at least be 50 cm with additional stitching when placed in the dry, and overlaps shall be staggered by minimum 150 cm.

Overlapping of geotextiles under water shall be as follows:

Water Depth [m]	Overlapping [m]
2.0	> 1.0
5.0	> 1.5
9.0	> 2.0

Overlaps are to be controlled by divers immediately prior to placing the cover layer and the Contractor in consultation with the Employer shall rectify any imperfection.

Geotextiles are to be covered with suitable materials as soon as possible, but within one week of being laid, but geotextile filter mats placed under water shall be covered immediately. When laying the covering material it shall not be dropped in the dry from a height greater than 1.5 m.

When geotextiles are installed on a slope, the cover material shall be deposited starting from the toe of the slope.

During installation of geotextiles and subsequent placing of cover layers no high stresses shall be exerted on the geotextiles and at no situation shall a geotextile be pulled around or along sharp edges. The installation equipment must be suitably designed to fulfil this requirement.

Stock piles of materials shall not be placed on top of laid geotextiles.

No construction equipment shall work on the geotextiles without at least 300 mm of suitable material overlying the geotextile.

7.5 CUBICAL CONCRETE BLOCKS

7.5.1 General Description

Placing of cover layer materials on the slope of embankments, shanks of impermeable groynes etc. shall start only after the toe of the revetment has been laid completely. It shall start at the toe of the slope in horizontal layers working upwards in a careful manner so as to avoid disturbance / damage (or, in case of granular filter layers, misplacement of the previous layer, and mixing up, washing out, disintegration, sliding, etc.). The method of placing has to ensure a minimum of voids, but a maximum of interlocking of the cover layer components.

Single blocks of a revetment are placed directly on the filter layer with no direct connection to adjacent the blocks, except by interaction. Thus, the stability of the cover layer is dependent on the stability of individual blocks, the size of which may be determined using the formulas of Chapter 5 and Design Plate A for current attack and wave attack respectively.

Above the water level concrete blocks shall be laid by hand on the filter layer in rows parallel to the direction of the flow. The blocks in each row shall be staggered half a block width from those in the previous rows. Adjacent blocks in the same row shall be laid with a gap between them of 10 mm if the block size is less than 50 cm, and of 20 mm if the block size is 50 cm or more.

Filling the space between adjacent blocks with granular material can enhance the stability of the cover layer. This has the effect of developing friction between the blocks provided that the granular material remains in place. Since the blocks are not connected, replacement of individual blocks and thus maintenance of the revetment is easy.

Below the water level concrete blocks shall be laid by controlled dumping with a nominal voids ratio of 35%. The nominal thickness of revetment cover layers shall be achieved over at least 75% of the area and nowhere shall the coverage be less than 80% of that as per specification.

A regular inspection and maintenance programme is recommended.

7.5.2 General Material Description

Cement shall be Ordinary Portland Cement complying with BS 12 or equivalent standard and fine and coarse aggregates shall comply with BS 882: Part 2.

Fine aggregates shall be clean natural sand free from injurious amounts of silt, clay, salt, organic or other harmful impurities.

Coarse aggregates for concrete blocks to be used in cover layers or falling aprons of a revetment shall be broken first class bricks, which are sound, hard and well burnt, uniform in size, shape and colour, homogeneous in texture and free from flaws and cracks. The minimum compressive strength of bricks should be 15 N/mm^2 and the maximum water absorption, which is the increase in weight after absorption in water for one hour, shall not be more than 16 %.

The water used for concrete mixing and curing shall be fresh water, clean and free from any substance injurious to the finished product. It shall meet the requirements of BS 3148 or equivalent.

The mix proportions shall be 1:3:6 and the cement content at least 260 kg for one m^3 ready mixed concrete. The free water to cement ratio shall be within 0.45 to 0.55 by weight.

During the production of concrete blocks quality tests shall be performed in accordance with relevant standards. The minimum compressive strength of each test cube after 28 days shall be 15 N/mm^2 and the minimum average compressive strength of each series of three test cubes shall be after 28 days 20 N/mm^2. The size of the test cubes shall be 20x20x20 cm.

Compaction of the concrete shall be properly done to secure maximum density and strength and the curing shall begin as soon as the concrete is sufficiently hard and shall be continued for 10 days.

7.5.3 Technical Specification for Cubical Concrete Blocks

7.5.3.1 General

Precast cement concrete blocks (cc-blocks) shall be of concrete Class B 15, DIN 1045 (crushing strength $\geq$ 15 N/mm^2 after 28 days) and shall be made to the dimensions specified on the Drawings and the Bill of Quantities, and to the tolerances specified in Section 7.5.3.9. The materials and workmanship shall comply with the Specification herein in all respects.

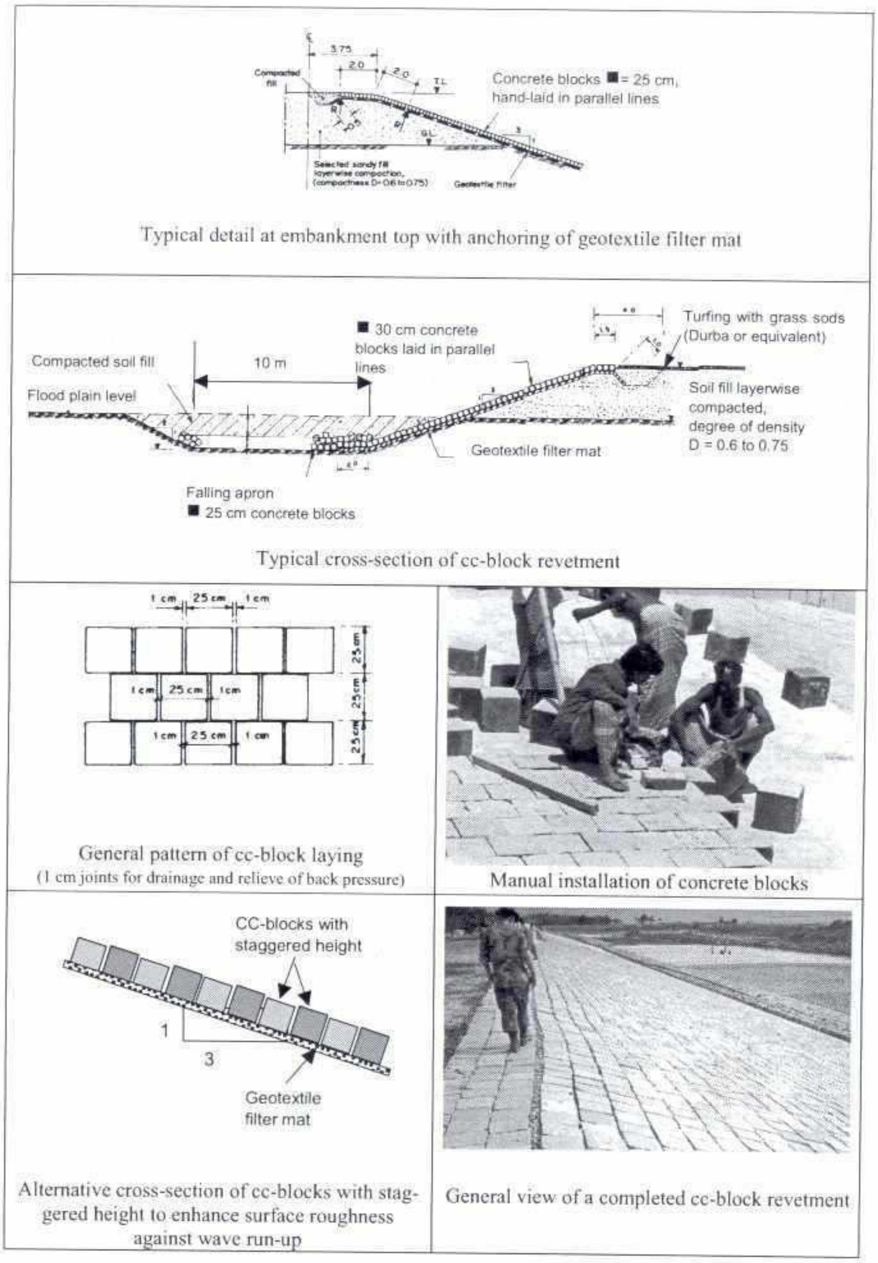
The Contractor may, subject to the approval of the Employer, modify the geometry of the large blocks slightly to facilitate its handling and placing e.g. by incorporating cylindrical holes or horizontal recesses. However, the finished weight of the blocks shall not be altered by such modification.

Blocks shall not be stockpiled until they have been cured in accordance with Section 7.5.3.9 hereof. They shall not be placed in the Works until at least twenty-eight days after casting have elapsed or the specified strength has been achieved.

Blocks, which are damaged during stockpiling, transport or handling, will be rejected by the Employer and shall be removed from the Site by the Contractor.

7.5.3.2 Cement

Cement shall be Ordinary Portland Cement complying with BS 12 or equivalent standard. A works test certificate must accompany each cement consignment delivered to the Site. In the absence of certificates, samples of cement shall be taken from the consignment on arrival at Site, in the presence of the Employer. These shall be forwarded for analysis and testing to a laboratory acceptable to the Employer.



Typical detail at embankment top with anchoring of geotextile filter mat

Typical cross-section of cc-block revetment

General pattern of cc-block laying
(1 cm joints for drainage and relieve of back pressure)

Manual installation of concrete blocks

Alternative cross-section of cc-blocks with staggered height to enhance surface roughness against wave run-up

General view of a completed cc-block revetment

Figure 7.5-1: Details of concrete block revetments

Cement may be delivered in sealed bags and shall be stored in a heat and moisture-protected and well-ventilated store. Each cement supply shall be stored separately and the cement shall be consumed in the sequence of its arrival at Site.

Cement, which is damp or contains lumps, which cannot be broken to original fineness by finger pressure, must be condemned irrespective of age and must be removed from the Site.

7.5.3.3 Concrete Aggregate - General

Fine and coarse aggregates shall comply with BS 882: Part 2. Testing of aggregates shall be in accordance with BS 812.

Approval of a source of aggregate by the Employer shall not be construed as constituting the approval of all materials to be taken from that source and the Contractor shall be responsible for the specified quantity and quality of all such materials used in the Works. The Contractor shall not obtain aggregates from sources that have not been approved by the Employer.

The Contractor shall provide appropriate means of storing the aggregates at each point where concrete is made, such that

- each nominal size of coarse aggregate and the fine aggregate shall be kept separated at all times;
- contamination of the aggregates by the ground or other foreign materials shall be effectively prevented at all times, and
- each heap of aggregate shall be capable of draining freely.

The Contractor shall make available to the Employer such samples of the aggregates as he may require. Such samples shall be collected at the point of discharge of aggregate to the batching plant or mixing machines. If any such sample does not conform to the Specification, the aggregate shall promptly be removed from the Site and the Contractor shall carry out such modifications to the storage arrangements as may be necessary to secure compliance with the Specification.

7.5.3.4 Fine Aggregates

Fine aggregates shall be clean natural sand of sharp angular grains of silica and the grains shall be hard, strong and durable. Fine aggregates shall be free from injurious amounts of silt, clay, salt and organic or other harmful impurities. The fineness modulus shall be 1.5 to 2.0.

The grading of fine aggregate for concrete shall be within the following range:

BS 410	150 ηm	300 ηm	600 ηm	1.18 mm	2.38 mm	5.0 mm
Standard Mesh	No. 100	No. 50	No. 30	No. 16	No. 8	No. 4
% of passing	0-10	20-35	35-70	65-85	85-100	95-100

Table 7.5-1: Grading for fine aggregate for concrete

7.5.3.5 Coarse Aggregates

Coarse aggregates for concrete blocks to be used in cover layers or falling aprons shall be broken first class bricks or so-called "picked Jhama" bricks.

Picked Jhama bricks shall be over-burnt first class bricks, uniformly textured, with good shape, slightly black in color and without cracks and spongy areas. Otherwise picked Jhama bricks shall meet the same Specification as first class bricks.

First class bricks shall be sound, hard and well burnt, uniform in size, shape and colour, homogeneous in texture and free from flaws and cracks. A fractured surface shall show a uniform compact structure, free from holes, lumps or grit. Minimum crushing strength shall be 15 N/mm2. Maximum water absorption shall be limited to an increase in weight after absorption in water for one hour by not be more than 16 percent.
Coarse aggregates shall be well graded within the following range, to the satisfaction of the Employer:

BS 410	75 mm	37.5 mm	20 mm	10 mm	5 mm
% of passing	100	95-100	35-70	10-40	0-5

Table 7.5-2: Grading for coarse aggregate for concrete

7.5.3.6 Water

The water used for concrete mixing, curing, or other designated applications shall be fresh water, clean and free from oil, salt acid, alkali, sugar, vegetable or any other substance injurious to the finished product. The water shall meet the requirements of the Standards, in particular DIN 4030 or BS 3148.

The Contractor as per BS 3148 shall examine water obtained from a source other than a public piped supply of potable water with regard to its suitability for producing concrete. The certificates are to be submitted to the Employer.

7.5.3.7 Nominal Concrete Mix for Blocks

Concrete for concrete blocks shall correspond to concrete Class B 15, DIN 1045, with the following requirements (tested at cube specimen of size 20 x 20 x 20 cm):

- minimum compressive strength after 28 days of each test cube: 15 N/mm²
- minimum average compressive strength after 28 days of each series of three test cubes: 20 N/mm².

The mix proportions shall be 1:3:6 and the cement content shall be at least 260 kg for one m³ of ready mixed concrete.

The free water to cement ratio shall be within 0.45 to 0.55 by weight.

7.5.3.8 Quality Control

During production of concrete blocks quality tests shall be performed in accordance with DIN 1045 or BS.

Six test cubes of 20 x 20 x 20 cm shall be prepared on every day of block production for each 100 m^3 of concrete poured to verify the compressive strength. In case more then one batching plant is supplying material for CC-block production, one set of six test specimen is to be produced for each plant on every working day. Alternatively cylindrical test specimen may be used.

Three each of each set of the test specimen shall be tested after 7 days and 28 days of its production. The compressive strength shall at least correspond to the minimum values stipulated under Section 7.5.3.7.

7.5.3.9 Production

Formwork and moulds shall ensure maintaining designed shapes and block sizes. They shall preferably of steel.

Mixing of concrete shall be done thoroughly to ensure that concrete of uniform color and consistency is obtained. Unless otherwise permitted by the Employer hand mixing of concrete is prohibited. Batching plants shall be used for mixing concrete.

The ingredients of concrete such as cement, fine aggregates, coarse aggregates and water shall be measured correctly for each batch of mixing. In case of volumetric batching the bulking of aggregates must be accounted.

Concrete shall be transported from the place of mixing to the place of final deposition as quickly as possible. The methods adopted should ensure that concrete is placed in position within 45 minutes, so that the evaporation and segregation of the mix is prevented. Rehandling shall not occur.

Concrete shall be placed directly in its final position avoiding segregation. Concrete should be placed gently at its position and not thrown from a height. Before placing concrete the formwork and moulds shall be cleaned and well wetted, to the satisfaction of the Employer.

Compaction of concrete shall be properly done to secure maximum density and strength. It shall be done immediately after placing of concrete.

Curing shall begin as soon as the concrete is sufficiently hard and shall be continued for 10 days. Curing methods may be by spraying water to the concrete, or by covering the concrete surface with a layer of gunny bags, canvas, hessian, straw or similar absorbent materials which is to be kept constantly wet, or another method approved by the Employer.

The ready concrete blocks shall not deviate by more than ± 5% by weight from the size specified on the Drawings or in the Bill of Quantities.

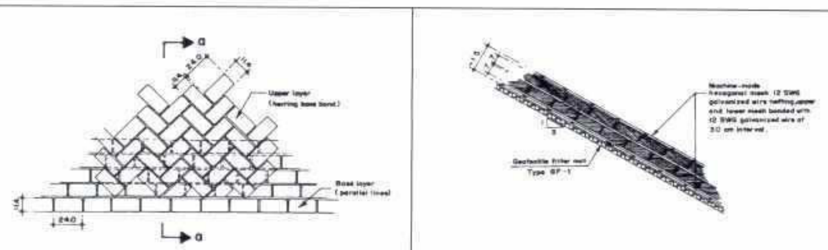
7.6 BRICK MATTRESSING

7.6.1 General Description

Brick mattressing is to be made of two layers of first class bricks, encased in double layer of 12 SWG galvanised or PVC-coated wire netting of less than 100 mm hexagonal mesh size, and with 12 SWG galvanised tie wires, bonding firmly the upper and lower wire mesh layer, at maximum 30 cm centre to centre spacing.

Brick mattressing is to be firmly supported by a footing of rubble or concrete blocks, which must be completed prior to laying any mattress.

The bottom layer of bricks shall be laid flat on the filter in rows parallel to the direction of current flow, with tight joints. The second layer shall be laid in herringbone bond. Thereby, the bricks of the second layer shall be laid flat for 15 cm thick mattresses, but in upright position for 20 cm thick mattresses



Plan view of typical brick mattressing (d = 15 cm)

Cross-section of 15 cm thick brick mattressing

Figure 7.6-1: Details of brick mattressing

7.6.2 Material Specification

Bricks to be used in mattresses for cover layers shall be first class bricks, sound, hard, well burned, uniform in size and colour, homogeneous in texture, well shaped with sharp edges, with even surfaces and without cracks, spongy areas, rain spots or flaw of any kind.

A fractured surface shall show a uniform compact structure, free from holes, lumps or grits.

In the water absorption test the increase in weight shall be less than 16% after deposition in water for one hour.

The brick dimensions shall be 240 x 120 x 70 mm, with ± 5 mm tolerance for any of the dimensions.

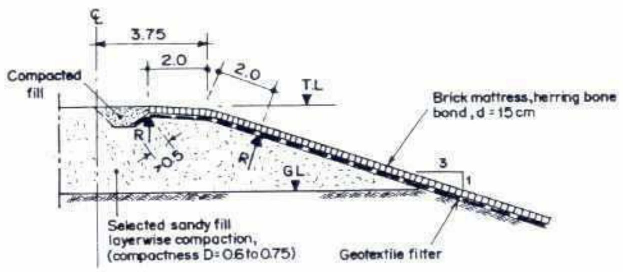
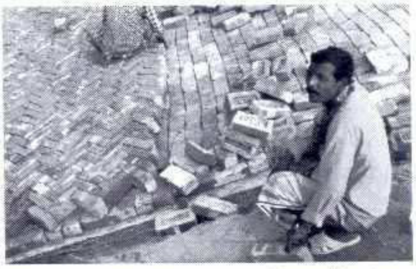


Figure 7.6-2: Typical detail at embankment top with anchoring of geotextile filter mat


Placing of bottom brick layer in flat position


Anchor trench at embankment top


Placing of upper brick layer in upright position (mattress thickness 18 cm)


Placing of toe protection and support for brick mattressing


Completed brick mattressing with toe protection

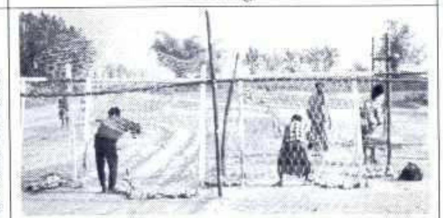

Site manufacture of wire mesh for brick mattressing

Figure 7.6-3: Brick mattressing works

7.7 RIP-RAP AND BONDED STONES

7.7.1 General Description

7.7.1.1 Rip-rap

One of the most common type of protective cover layer is rip-rap, which generally comprises well graded and randomly placed stone or rock. It has the following advantages:

- easily to place, also under water;
- high hydraulic roughness to alternate currents and waves;
- flexibility;
- low maintenance requirements and convenience to repair;
- self-repairing to some extent;
- durability, and
- environmentally acceptable appearance.

The nominal size and the grading range of the material depend on the hydraulic loads and can be determined by application of the relevant formulas in Chapter 5 and Design Plate B for current attack and wave attack respectively.

The specific gravity of the stones can vary from 2.0 t/m^3 for sedimentary rocks up to 2.9 t/m^3 for igneous rocks, but is typically in the range 2.5 to 2.7 t/m^3.

The best suited are angular, nearly cubical stones. The ratio of the maximum to the minimum dimension of a stone should be less than 3 to 4. It is recommended that typical ratios for bank protection should be less than 2.5 for 70 % of the stones, less than 2.0 for 85 % and less than 2.5 for all stones.

The mixture of stones should form a smooth grading curve and for optimisation of the stability of the cover layer it is recommended to keep the gradation within the limits shown in the respective grading diagrams, with a coefficient of uniformity $C_u \leq 2.15$.

Rip-rap protective layers can be constructed without special equipment. It can be dumped on slopes not exceeding 1:2 or hand-placed on slopes not steeper than 1:1.5, but the recommended slope is 1:3.

Special care must be taken in case of geotextile filter layers, which can be damaged by the angular shape and sharp edges of the rip-rap material. It is therefore recommended to use either a geotextile resistant to localised stresses or to provide a granular sublayer of smaller stones or gravel to act as a protective layer.

When placing rip-rap on an under water slope by dumping, the material can become segregated depending on the flow velocity, the water depth and the stone sizes. Therefore, it should be released as close as possible to its final position. Installation of the latter, however, is only possible in areas with limited flow velocities and requires relevant experience of the contractor.

To enhance the stability of rip-rap against hydraulic forces, bonding by colloidal cement grout or sand-bitumen mix are suitable and easy to apply methods. For a given situation the nominal size of rip-rap stone may be reduced if bonding is provided, as compared to loose rip-rap structure.

7.7.1.2 Bonded Rip-Rap

In order to improve the stability of a cover layer and hence of the whole revetment, rip-rap and pitched stones can be bound with colloidal cement grout or bitumen. There are 3 basic methods of grouting:

- Surface grouting fills about 30 % of the surface voids over the whole area. The grout penetrates the surface layer, but the cover layer is not completely sealed;
- Pattern grouting fills 50 to 80 % of the voids. The whole thickness of the cover layer is penetrated with an appropriate pattern;
- Full grouting fills 100 % of the cover layer voids resulting in an impermeable and heavy cover layer.

To provide a more permeable revetment, it is recommended that not more than 70 % of the surface should be grouted. When the permeability is reduced, the stability of the cover layer may be adversely affected by excess pore water pressure. In such cases weepholes should be provided to avoid a massive failure.

Surface grouting may be given consideration when deciding the nominal stone size for a revetment protection. Generally a reduction of the nominal stone size for the stability under wave attack by up to 10 % may be considered, for pattern grouting up to 40 %.

7.7.2 Technical Specifications

7.7.2.1 Stone Fill (Rip-Rap)

(a) General

Within this Section "stone" shall include natural stone material obtained from rivers or quarries. The material supplier shall satisfy himself that the sources of materials in Bangladesh or abroad have sufficient capacity to meet the requirements of the project. In the context of capacity, both the quantity of production and its compliance with the delivery schedule must be taken into account.

(b) Material Standard and Tests

The material shall be free from dirt, sand, gravel and materials of organic origin (roots etc.), and shall be good sound stone / rock without cracks/fissures to avoid breaking during handling and placing or dumping.

General view of a completed rip-rap revetment

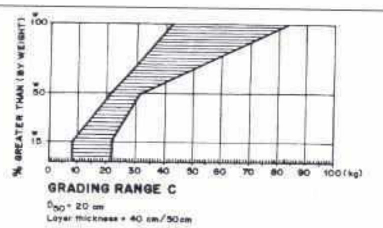
Surface grouting

Pattern grouting

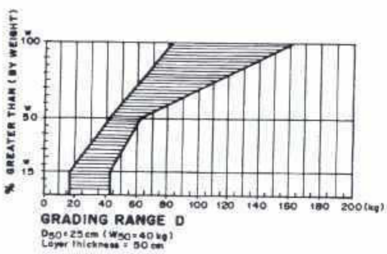
Full grouting

(Source: PIANC, 1992)

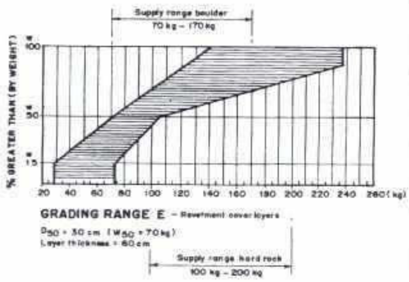
Revetment of rip-rap placed on a granular filter



Rip-rap Range C
(suited for pattern grouting by bitumen grout)



Rip-rap Range D
(suited for surface grouting by colloidal cement grout)



Rip-rap Range E
(suited for loosely placed stones / boulders and quarry rock)

Figure 7.7-1: Rip-rap and bonded stones

The following standards shall apply:

- BS 812: Testing aggregates
- ASTM C 88: Aggregate soundness
- ASTM C 535: Abrasion resistance

The following test result shall be verified and respective reports and certificates submitted:

- The weight average loss shall not exceed 10% by weight, when subjected to the ASTM C-88 Sodium Sulphate Soundness Test.
- The average bulk specific gravity of any sample shall be in the range of 2600 kg/m^3 (BS 812, Part 2, Chapter 6, 3).
- Water absorption of rock material shall not exceed 6 % (BS 812, Part 2, Chapter 5).
- Minimum compressive strength as per ASTM C 170-50: 100 N/mm^2.
- The percentage of wear shall no exceed 40% (ASTM C 535), when subjected to the Los Angeles Test.

Only stone / rock with a factor not exceeding 2.5 between the longest and shortest dimension of the rock shall be allowed in the delivery.

(c) Grading Tests at Point of Delivery

Testing at the agreed point of delivery (stockpile yard) shall take place at the supplier's expenses. The location for testing shall be arranged by the Contractor and approved by the Employer.

The gradation of materials stock piled at the yard shall be tested at least one time for each 500 t of delivery.

Samples for determination of weight gradation shall contain at least 100 individual stones / rocks. The samples shall be taken by random selection from each specified gradation to obtain representative samples, and shall confirm to the grading shown in Table 7.7-1 to Table 7.7-3.

Should tests show non-compliance with the specified gradation range the Employer may order more tests to be performed. In case the additional tests show non-compliance the Employer may reject whole or part of the rock delivery.

Rejected material shall be removed from the stock pile yard or blended with suitable rock sizes by the Contractor at his own expense, all to the approval of the Employer.

<u>7.7.2.2 Cement-Grouted Rip-Rap Layers</u>

In designated areas, rip-rap cover layers may be bonded by sand-cement grout to secure the stability and integrity of the revetment cover layer. Cement grout or sand-cement grout to be used for bonding stones in cover layer is defined to be only <u>colloidal grout</u>.

ນຈລ

Range C	% Smaller than (by weight) [kg]			Stone Sizes [cm]		
	W_{15}	W_{50}	W_{100}	D_{15}	D_{50}	D_{100}
min.	8.3	20.8	41.6	15	20	25
max.	21.8	31.2	83.2	20	23	32

Table 7.7-1: Grading Range C
(suited for rip-rap layers with bitumen pattern grouting)

Range D	% Smaller than (by weight) [kg]			Stone Sizes [cm]		
	W_{15}	W_{50}	W_{100}	D_{15}	D_{50}	D_{100}
min.	16.3	40.6	81.3	18	25	32
max.	42.7	60.9	162.5	25	29	40

Table 7.7-2: Grading Range D
(suited for rip-rap layers with surface grouting by colloidal cement grout grouting)

Range E	% Smaller than (by weight) [kg]			Stone Sizes [cm]		
	W_{15}	W_{50}	W_{100}	D_{15}	D_{50}	D_{100}
min.	28.1	70.2	140.4	22	30	38
max.	73.7	105.3	237.0	31	34	45

Table 7.7-3: Grading Range E
(suited for rip-rap layers without additional bonding)

In the above tables the following definition applies:
D_{50} = nominal size/diameter of stones
W_{15} = 15% of the stones of a gradation range may be less than the weight specified against W15;
W_{50} = 50% of the stones of a gradation range shall be at least of the weight specified against W50;
W_{100} = 100% of the stones of a gradation range shall not exceed the weight specified against W100.

The bonding is to be carried out as surface grouting, whereby, however, an adequate permeability of the cover layer has to be guaranteed by absolute controlling the quantity of grout placed per area. The cement grout has to be poured in such a way to ensure that all stones and boulders of the cover layer are well bonded and secured against displacement by hydraulic forces, including uplift forces. The quantity of grout to be applied depends on the volume of voids of the stone material.

For surface grouting of cover layers (D_{50} = 25 cm, Grading Range D, layer thickness $\geq$ 50 cm) above water level, where the stone material can be placed well and relative dense, at least 100 ltr. grout per one square meter of surface area are to be applied.

For above-water areas, the cement grout should be supplied to the surface to be grouted by pump, using flexible hoses. However, manual pouring by use of buckets is also possible as long as the quantity of grout placed can be properly controlled.

Transitions into adjacent test structures with loose stone cover layers shall be grouted with 50 ltr./m^2 for a width of 10 m.

Cement grout shall only be produced in a high-speed mixer with at least 2,200 rpm (so-called colloidal grout mixer).

The minimal cement content for colloidal grout is 550 kg/m^3. Only fresh cement is to be used and its quality is to be verified by the Contractor through factory test certificates or independent tests by a recognized material testing laboratory. The water-cement ratio is to be maintained at 0.6.

Clean sand of grading range 0-2 mm may be used as aggregate for the colloidal grout at a quantity of about 1,600 kg/m^3 grout.

The aggregate should comply with the following granulometric composition:

Grain size [mm]	Percentage passing the test sieve (by weight)
0.25	≤ 25
0.50	≤ 60
2.00	≥ 90
4.00	100

Table 7.7-4: Grading of aggregates for cement grouted rip-rap layers

The water used for cement grout mixing shall be fresh water, clean and free from oil, salt acid, alkali, sugar, vegetable or any other substance injurious to the cement grout. The water shall meet the requirements of DIN 4030 or BS 3148.

The Contractor prior to starting the work shall carry out trial mixes. Thereby the selected materials and mixing equipment must be the same as will be used subsequently at the Site.

The trials shall include:

- cement quality tests,
- grain size distribution tests of aggregates,
- consistency tests, and
- compressive strength tests.
-

The compressive strength shall be verified on test specimen of size 20 x 20 x 20 cm, and a comprehensive test report is to be submitted to the Employer for approval. The test results shall be 20% above the standard values stipulated in the following.

The standard compressive strength of the cement grout shall meet the following requirements when tested on test cubes of size 20 x 20 x 20 cm after 28 days of pouring.

- Characteristic strength (minimum compressive strength of each cube): 25 N/mm²
- Series strength (minimum compressive strength of each series of three cubes): 30 N/mm²

For each day on which grouting works are being carried out at least three test specimen (size 20 x 20 x 20 cm) are to be prepared.

The consistence of colloidal grout, when subjected to the flow table test as per DIN 1048/1 shall be in the following range:

- One minute after lifting the mould, without striking the flow table: diameter of grout spread on the table (mean value as measured in two directions): a = 32 ± 2cm
- When measured immediately after striking the flow table 15-times: diameter of grout spread on the table (mean value as measured in two directions): a = 48 ± 2 cm.

7.7.2.3 Bitumen-Grouted Rip-Rap Layers

In designated areas rip-rap cover layers may be bonded by pattern grouting, using bituminous mortar, to achieve a flexible but integral cover layer, which can withstand substantial hydraulic loading.

To maintain the required permeability of the revetment cover layer not more than 70% of the stone structure should be grouted.

Good mix composition and execution of the grouting work is essential to ensure that the grouting mortar does not remain in the top layer or sags to form a dense layer below. Therefore highest attention is to be paid to control the proper viscosity of the grout by adding appropriate amount of aggregate to the mix and to maintain the correct mix temperature up to the time of pouring the bituminous grout on to the stone cover layer.

For rip-rap cover layers of Grading Range C (D_{50} = 20 cm) the quantity of bituminous grout should be in the range of:

- layer thickness 40 cm : 90 to 110 ltr./m²
- layer thickness 50 cm : 120 to 140 ltr./m²

Trials are to be carried out at Site by the Contractor to determine the practical quantities of grout required in accordance with the realistic volume of voids of the placed stone cover layers, subject to Employer's approval.

The bitumen used as a binder for grout shall be penetration graded bitumen of penetration Grade 80 - 100 and shall meet the requirements of BS 3690.

The Contractor has to submit to the Employer a certificate of origin of the bitumen as well as respective factory test certificates or have bitumen samples tested at a recognized material testing institute, to prove compliance with the Standard.

Clean sand of grading range 0-2 mm is to be used as aggregate.

Grain size [mm]	Percentage passing the test sieve (by weight)
0.25	≤ 25
0.50	≤ 60
2.00	≥ 90
4.00	100

Table 7.7-5: Grading of aggregates for bitumen-grouted rip-rap layers

Filler shall consist of limestone dust or Portland cement. It shall be free from lumps and other objectionable material and its grading shall comply with the following:

Sieve size BS 410	Percentage passing
600 micron	100
180 micron	100 - 95
75 micron	100 - 80

Table 7.7-6: Grading of filler for bitumen-grouted rip-rap layers

The composition of bituminous grout shall be in the following range:

Component	Composition Percentage by Weight	
	Range	Suggested
Sand 0-2	60 - 70	67
Filler	10 - 30	14
Bitumen Grade 80/100	14 - 20	19

Table 7.7-7: Composition of bituminous grout

The Contractor in consultation with the Employer shall determine the optimal mix composition. A comprehensive report on trial mixes, including grading tests of aggregates and filler, bitumen tests, mixing methods, temperature and the recommended mix design is to be prepared by the Contractor and to be submitted to the Employer for approval.

For the quantity of bituminous grout to be produced for the pilot project the employment of an asphalt mixing plant cannot be justified. Therefore, traditionally customary methods may be used and the Contractor has to submit respective detailed proposals to the Employer for approval.

The following principle requirements have to be met:

- Separate and clean storage facilities are to be provided for bitumen, sand and filler, so that contamination with other materials cannot occur.
- The mixing facilities shall be arranged in a way that representative samples of the mix components can be taken.
- The temperature of bitumen and of the mineral aggregates is to be constantly controlled and recorded.
- The mixing facilities are to produce a homogeneous mixture of bitumen, aggregates and filler.

Aggregates are to be dried to such an extent that the moisture content has a maximum value of 0.1% (by weight), when supplying the aggregates to the mixing facilities. Respective tests are to be carried out by the Contractor in the presence of the Employer's Representative and results are to be recorded.

The heated mineral fractions shall have a minimum temperature of 140° Celsius and a maximum temperature of 220° C.

Bitumen, which has been heated to exceed a temperature of 190° Celsius, shall not be used for bituminous grout.

During transport of the mix from the mixing facilities to the place of pouring the bituminous grout is to be protected against adverse weather conditions.

When applying petroleum as an anti-adhesive to the means of transport the use shall be limited to uniformly applied quantities not exceeding 50 g/m^2.

Immediately prior to pouring the bituminous grout the application temperature shall be in the range of 100° to 140° Celsius, which is to be regularly controlled at the spot, in the presence of the Employer's Representative.

208

7.8 WIREMESH MATTRESSING

7.8.1 General Description

Wiremesh cages for mattress-type protective layers are containers made of galvanised or PVC-coated steel wire mesh. The cage thickness is depending on the hydraulic load and the fill material. Normally, they are filled with stones. The size of the stones and the thickness of the mattresses can be determined using the formulas of Chapter 5 and Design Plates C and D for current attack and wave attack respectively.

Wiremesh mattresses are very flexible elements. The thickness ranges between 15 cm and 36 cm for standardized structures SC2 or SC 3. They are used in a single layer to form a protective cover layer to the surface of a river- bank or an embankment, the slope of which is recommended at 1:3, but should not be steeper than 1:1.5.

Wiremesh mattresses are to be pre-assembled and, if installed above the water level, placed on the designated slopes or the floodplain. They have to be stretched out on the surface and any unnecessary creases must be stamped out. If the finished mattress is to be less than the full length, the required length is cut off.

The diaphragms must be perpendicular to the direction in which the filling will move, either down the slope or in the direction of the flow. Prior to filling, the individual cages must be connected to each other along all corners using the proper lacing wire.

When the mattress is placed on a geotextile filter, care must be taken to ensure that projecting ends of the wire are bent upward to avoid puncturing or tearing the filter cloth.

In case the mattresses have to form a curve, individual mattresses can be divided diagonally to form two triangular sections. The open side of one section is to be butt-jointed to the intact side of the next section. Placing whole mattresses between triangular sections can modify the curve.

The filler material must have a nominal diameter of about 1.5 times the mean mesh dimension and individual units should be greater than the nominal mesh size. Accordingly, the minimum stone size acceptable for such mattress fill should not be less than D = 10 cm, i.e. just larger than the wire mesh dimensions.

Brick fill of mattresses shall be carried out using only full-size and half-size bricks. Any fill material shall be properly and densely dumped in the mattress cages in order to fill the units to maximum extent with the minimum of voids. Particular attention is to be paid to neat filling at the mattress corners.

Partial or full grouting with asphalt mastic can increase the stability and durability of gabion mattresses.

Where mattresses are to be installed below the water level, individual mattresses are to be pre-assembled and joined to units, which are to be filled densely with the specified material. After closing the lid covers tightly, the complete unit is to be lifted by special appropriate means and to be laid on the prepared slope or bed. Proper and tight placing of the units must be ensured one closely to the other. This work requires employment of heavy crane equipment. This, in conjunction with the high executional risk when working under strong river flow conditions, concludes that use of such mattresses for under water application is not recommended for standardized structures SC 2 and SC 3.

7.8.2 Material Specifications

7.8.2.1 Wiremesh Cages

Wiremesh mattresses are large and thin box-type construction elements made of zinc plus PVC-coated hexagonal double twisted wire mesh (wire dia. 2 mm / 3 mm including coating layers). The mesh is stretched on steel bars with a diameter of at least 12mm, which form the edges of the mattress cages.

Mattress dimensions of 4 x 2 m are recommended, with partitions at every 1 m.

The nominal mesh size varies from 44 x 60 mm to 100 x 120 mm. The recommended mesh size is 60 x 80 mm.

The mattress thickness depends on the type of fill material and is selected to

- d = 23 cm (stone fill)
- d = 36 cm (full-sized brick-fill)
- d = 36 cm (stone fill, at exposed situations)

The wire mesh must comply with the following factory specifications:

- the wire used for the manufacture of mattress and the lacing wire shall have a tensile strength of 38-50 kg/mm^2 according to BS 1052/80 "Mild Steel Wire".
- Zinc coating at 240 g/m^2 meet the requirements of BS 443/32 and DIN 1548.
- PVC-coating conforms to ASTM and has a thickness of 0.5 mm.

7.8.2.2 Brick Fill Material

Bricks to be used for mattresses filling in cover layers shall be first class bricks, sound, hard, well burned, uniform in size and color, homogeneous in texture, well shaped with sharp edges, with even surfaces and without cracks, spongy areas, rain spots or flaw of any kind.

The brick dimensions shall be 240 x 120 x 70 mm, with ± 5 mm tolerance for any of the dimensions. Minimum crushing strength shall be 15 N/mm^2. In the water absorption test the increase in weight shall be less than 16% after deposition in water for one hour.

7.8.2.3 Stone Fill

The stones shall be good sound material without cracks/fissures to avoid breaking during handling and placing or dumping.

The following standards shall apply:

- BS 812: Testing aggregates
- ASTM C 88: Aggregate soundness
- ASTM C 535: Abrasion resistance

The following test result shall be verified and respective reports and certificates submitted:

- The weight average loss shall not exceed 10% by weight, when subjected to the ASTM C-88 Sodium Sulphate Soundness Test.
- The average bulk specific gravity of any sample shall be in the range of 2600 kg/m^3 (BS 812, Part 2, Chapter 6, 3).
- Water absorption of stone material shall not exceed 6% (BS 812, Part 2, Chapter 5).
- Minimum compressive strength as per ASTM C 170-50: 100 N/mm^2.
- The percentage of wear shall no exceed 40% (ASTM C 535), when subjected to the Los Angeles Test.

Only stones with a factor not exceeding 2.5 between the longest and shortest dimension of the stone shall be allowed in the delivery.

7.8.2.4 Grading Tests at Point of Delivery

Testing at the agreed point of delivery (stockpile yard) shall take place at the supplier's expenses. The location for testing shall be arranged by the Contractor and approved by the Employer.

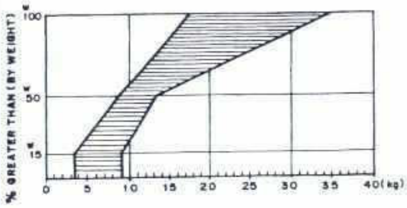
The gradation of materials stock piled at the yard shall be tested at least one time for each 500 t of rock delivery.

Samples for determination of weight gradation shall contain at least 100 individual rocks. The samples shall be taken by random selection from each specified rock gradation to obtain representative samples, and shall confirm to the following Table 7.8-1 and Fig. 7.8-1.

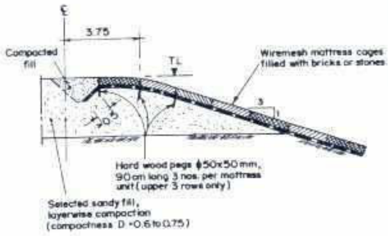
Range B	% Smaller than (by weight) [kg]			Stone Sizes [cm]		
	W_{15}	W_{50}	W_{100}	D_{15}	D_{50}	D_{100}
min.	3.5	8.8	17.6	11	15	19
max.	9.2	13.2	35.1	15	17	24

Table 7.8-1: Stone ranges for fill of wiremesh mattresses

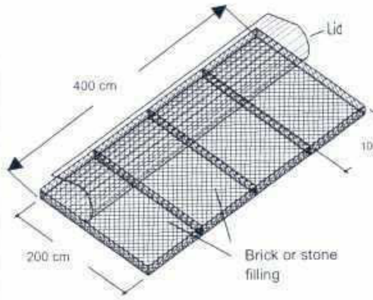
View of completed revetment with stone-filled wiremesh mattresses



Stone fill – Grading Range B – D_{50} = 15 cm



General detail of wiremesh mattress arrangement as revetment of upper slope zone

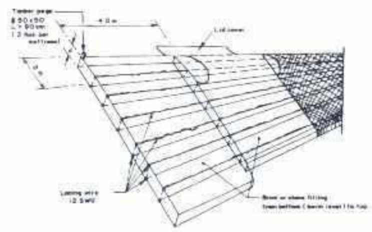


Typical design of local-made wiremesh boxes (each compartment framed with welded ϕ 12 mm plain reinforcing bars)

Box height (for SC 2 and SC 3):

- Brickfill h = 36 cm
- Stone fill (D_{50} = 15 cm) h = 25 cm
- Stone fill (exposed edges) h = 36 cm

Installation and brick-filling of local made wiremesh boxes (d = 36 cm)



Typical arrangement of wiremesh boxes along the embankment slope

Figure 7.8-1: Wiremesh mattressing

7.9 INTERLOCKING CC-SLAB UNITS

7.9.1 General Description

In a drive to reduce the cost for standardized revetment cover layers various shapes of concrete slabs, which are interlocking, have been developed to provide more positive interconnection between the individual blocks. The interlocking gives an additional stability by including the weight of adjacent blocks and thus the weight of the individual blocks can be reduced. However, the increased stiffness of the cover layer in comparison with loose blocks can lead to the bridging of small scour holes or rain cuts, which may have unnoticeable dangerous dimensions and collapse.

Two popular systems are rectangular slabs with ship-lap type joints respectively tongue-and-groove joints. The stability of both types depends among other things on the strength and durability of the interlock, because the block would not be stable by its weight alone. The production of such slabs requires high accuracy, wherefore such systems should be factory-made. It is a further disadvantage that such slabs are susceptible to damages during transport and handling.

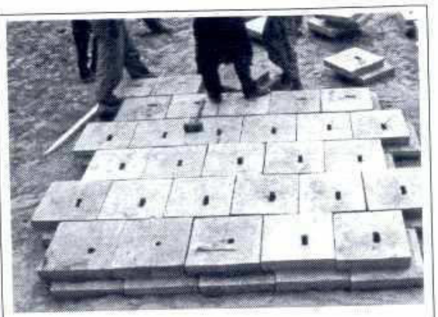
Interlocking slabs are generally placed by hand. They are suited only for installation above the water level, and are not suited for laying at sharp curvatures, e.g. at termination areas. Replacement of individual slabs within maintenance services can be difficult.

Based on the experience gained within the FAP 21 pilot project these slab-type protection systems are not yet recommendable due to various implementation risks involved. However, with a view to their cost-effectiveness, it is worthwhile to further improve these systems.

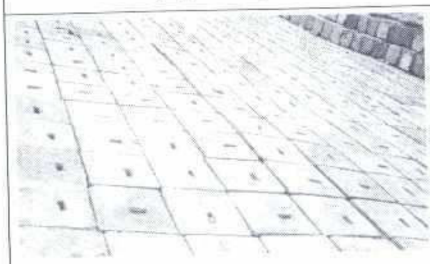
7.9.2 Material Specifications

Interlocking cement concrete slabs shall be factory-made, using block-making machines or vibrator tables. The concrete quality shall be to Class B 45, DIN 1045, corresponding to a concrete compressive strength of at least 45 N/mm^2. Cement shall be Ordinary Portland Cement complying with BS 12 or equivalent standard. Minimum cement content should be 350 kg for one m^3 of ready mixed concrete. Concrete aggregates shall be sand and gravel, respectively crushed stone, but Khoa shall not be used under any circumstances. All fine and coarse aggregates shall comply with BS 882: Part 2 and be clean and free from injurious amounts of silt, clay, salt, organic or other harmful impurities. Aggregate grain size distribution should be 0 – 16 mm. The water used for concrete mixing and curing shall be fresh water, clean and free from any substance injurious to the finished product. It shall meet the requirements of BS 3148 or equivalent. The free water to cement ratio shall be maximum 0.45 by weight. During the production of concrete blocks quality tests shall be performed in accordance with relevant standards. The minimum compressive strength of each test cube after 28 days shall be 45 N/mm^2 and the minimum average compressive strength of each series of three test cubes shall be after 28 days 50 N/mm^2 with test cubes size 20x20x20 cm. Compaction of the concrete shall be properly done to secure maximum density and strength and the curing shall begin as soon as the concrete is sufficiently hard and shall be continued for 10 days.

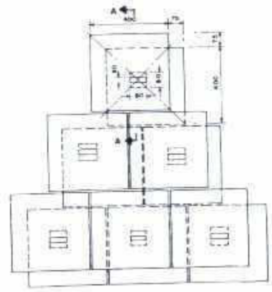
Handling of ship-lap slabs for placing on the prepared slope



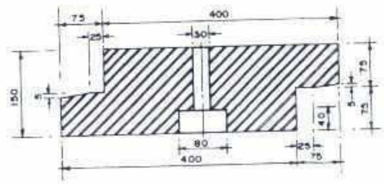
Placing pattern for ship-lap slabs



General view of completed protective layer



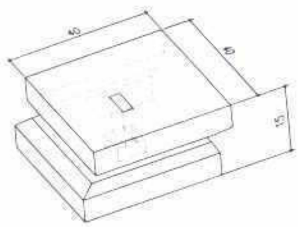
Installation pattern for ship-lap slabs



Cross-section of ship-lap slab

Recommended thickness:

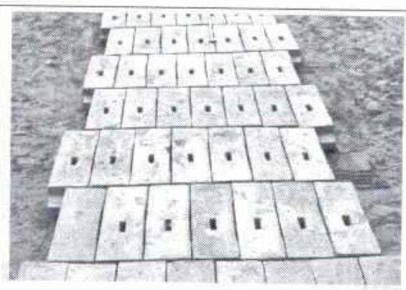
- Structure Class SC 2: 150 mm
- Structure Class SC 3: 200 mm



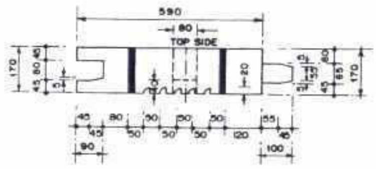
Isometric view of a single ship-lap slab (for SC 2)

Figure 7.9-1: Ship-lap type cement concrete slabs

Handling of groove-tongue slabs for placing on the prepared slope



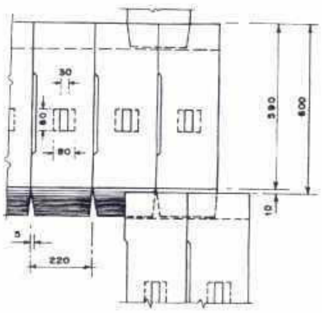
Placing pattern for groove-tongue slabs



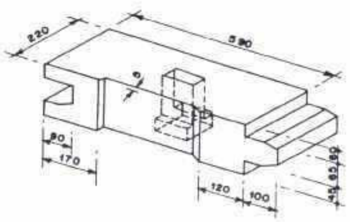
Cross-section of groove-tongue slab

Recommended thickness:

- Structure Class SC 2: 170 mm
- Structure Class SC 3: 220 mm



Placing pattern for groove-tongue slabs



Isometric view of a single groove-tongue slab
(dimensions suiting SC 2)

Figure 7.9-2: Groove tongue cement concrete slabs

202

7.10 LAUNCHING APRONS

7.10.1 Articulating CC-Block Mattress System (ACM)

7.10.2.1 General Description

Articulating cc-block mattress systems (ACM) consist of in-situ cast cement concrete blocks fixed to a geotextile filter-cum-bearing mat, and which are tied together cross-wise by multi-wire steel cables. The main advantages of such an integrated mattress system are:

- articulating behavior
- enhanced flexibility and stability over individual protective elements
- improved sustainability against erosion processes
- reduced risk of localized failure
- ease of construction in a dry construction pit
- minimized problems regarding vandalism and theft

A heavy geotextile filter mat represents the base of the mattress. The filter mat must posses a very high tensile strength and elongation behaviour due to the anticipated system deformations once the erosion process starts. The material should conform at least to the properties specified in Section 7.4.2.3.

The cement concrete blocks are cast directly onto the filter mat and must be connected to the mat by U-shaped steel needles.

To ensure integrity of the system, parallel and perpendicular steel wire cables are run through the in-situ cast blocks. All cables running perpendicular to the embankment slope are to be anchored to deadmen at the base of the upper revetment.

The block size is to be determined using the formulas of Chapter 5 and Design Plate E for current attack and wave attack respectively. The cabling should not be taken into account for determining the individual block sizes, but they are provided to enhance the system stability in case of extreme conditions from non-anticipated hydraulic forces or excessive riverbed erosion processes.

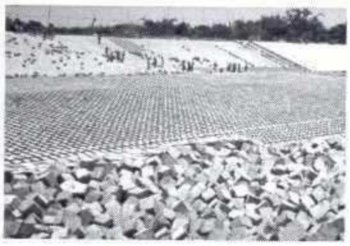


Figure 7.10-1: Overview of ACM-launching apron with falling apron

The cables should be multi-core steel, but not from synthetic fibres. The latter are plastic based, and their durability as well as excessive elongation under load rule out their use. Furthermore, they are more vulnerable to abrasion and vandalism.

Generally, such ACM would suit installation below water as well. Mattress units of appropriate dimensions may be produced on-shore. For site handling, transport and placing suitable steel frame arrangements are to be provided, to which the ready-made mattress can be suspended. Heavy crawler cranes with large out-reach are required, on-shore as well as off-shore. Such heavy equipment is expensive, and may even not be available from common sources. In addition, the flow conditions with the major rivers of Bangladesh would substantially increase the construction risks; even prevent the execution at times. Therefore, the use for under-water installation is not recommended for Structure Categories SC 2 and SC 3.

7.10.2.2 Technical Specifications

(a) Concrete Blocks for ACM

Cast in-situ cement concrete blocks for ACM shall be of concrete Class B 25, DIN 1045 (crushing strength ≥ 25 N/mm^2 after 28 days) and shall be made to the dimensions specified on the Drawings The materials and workmanship shall comply with the Specification herein in all respects.

Cement

Cement shall be Ordinary Portland Cement complying with BS 12 or equivalent standard. A works test certificate must accompany each cement consignment delivered to the Site. In the absence of certificates, samples of cement shall be taken from the consignment on arrival at Site, in the presence of the Employer. These shall be forwarded for analysis and testing to a laboratory acceptable to the Employer.

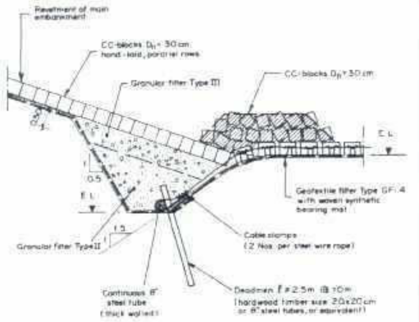
Cement may be delivered in sealed bags and shall be stored in a heat and moisture-protected and well-ventilated store. Each cement supply shall be stored separately and the cement shall be consumed in the sequence of its arrival at Site.

Cement, which is damp or contains lumps, which cannot be broken to original fineness by finger pressure, must be condemned irrespective of age and must be removed from the Site.

Concrete Aggregates

Fine and coarse aggregates shall comply with BS 882: Part 2. Testing of aggregates shall be in accordance with BS 812.

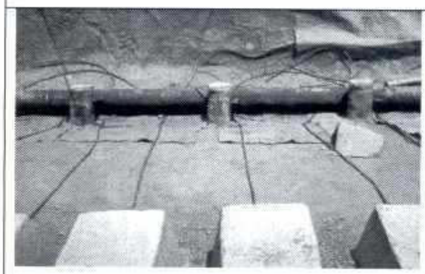
Approval of a source of aggregate by the Employer shall not be construed as constituting the approval of all materials to be taken from that source and the Contractor shall be responsible for the specified quantity and quality of all such materials used in the Works. The Contractor shall not obtain aggregates from sources that have not been approved by the Employer.



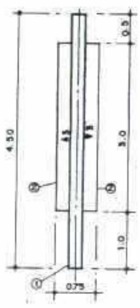
ACM anchorage at the base of upper revetment (berm level)

Installation of deadmen for ACM anchorage by guided drop weight

View along anchor trench at berm level

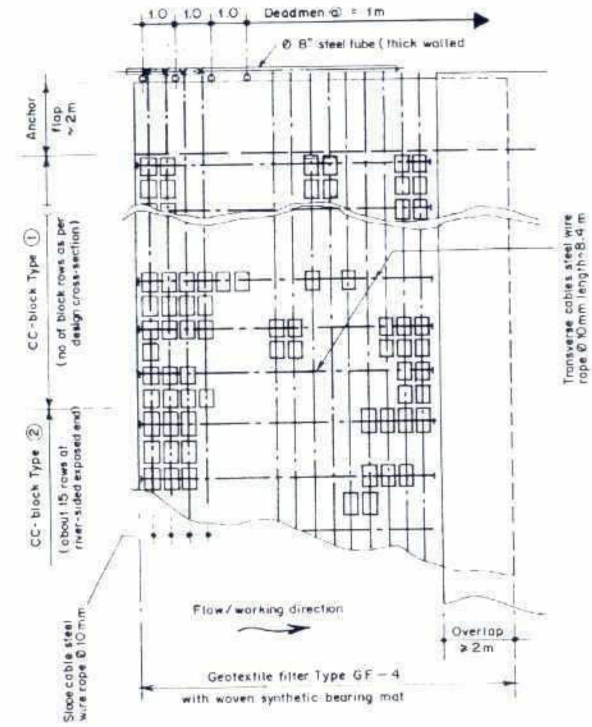


Front view against deadmen and anchorage cross-beam

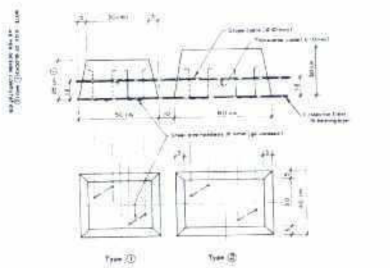


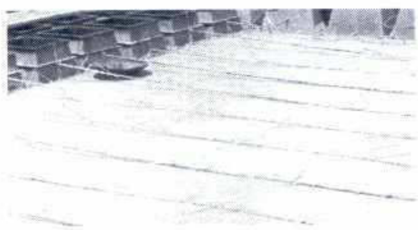
Deadman design
(steel pipe dia. 150 mm)

Figure 7.10-2: Anchoring of ACM (articulating concrete block mattress) for application as launching apron in a revetment structure



General layout of ACM for Structure Category SC 2





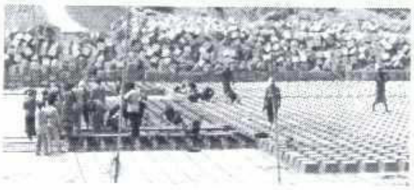
Concrete block design

- Type 1 for SC 2 (slope cables Ø ≥10 mm)
- Type 2 for SC 3 and at transition to falling aprons (slope cables Ø ≥15 mm)

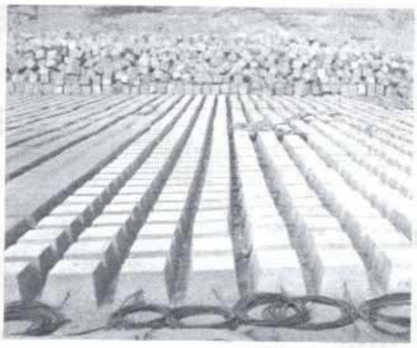
Geotextile bearing mat with block needles and anchor cables (construction stage)

Figure 7.10-3: Details of ACM CC-block mattresses

Preparation of moulds for casting cc-blocks



Casting cc-blocks for an ACM



View of ACM from (uncompleted) berm anchorage towards river-side (randomly placed cc-blocks as falling apron)

Figure 7.10-4: Installation of ACM CC-block mattresses

The aggregate mix for concrete shall be as coarse and dense as possible and shall be within the limits of grading curves "A 16" and "B 16" respectively "A 32" and "B 32".

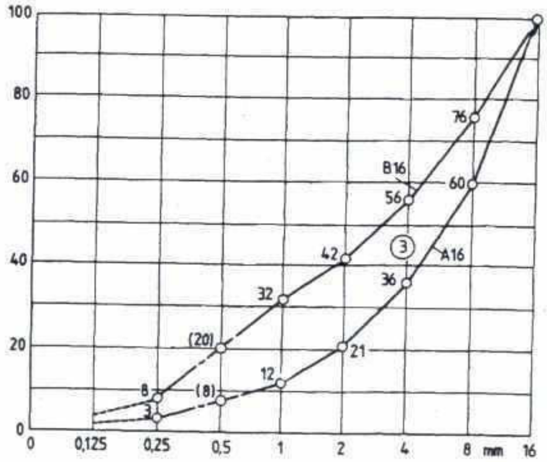


Figure 7.10-5: Grading curves of concrete aggregates for ACM (range A 16 and B 16, DIN 1045)

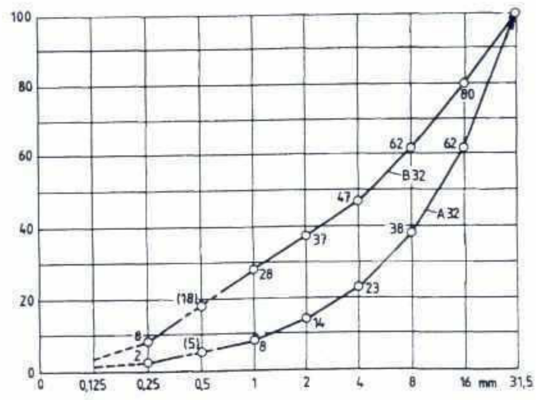


Figure 7.10-6: Grading curves of concrete aggregates for ACM (range A 32 and B 32, DIN 1045)

The Contractor shall provide appropriate means of storing the aggregates at each point where concrete is made, such that

- each nominal size of coarse aggregate and the fine aggregate shall be kept separated at all times;
- contamination of the aggregates by the ground or other foreign materials shall be effectively prevented at all times.

The Contractor shall ensure that graded coarse aggregates are tipped, stored and removed from store in a manner that does not cause segregation.

The Contractor shall make available to the Employer such samples of the aggregates as he may require. If any such sample does not conform to the Specification, the aggregate shall promptly be removed from the Site and the Contractor shall carry out such modifications to the storage arrangements as may be necessary to secure compliance with the Specification.

Water

The water used for concrete mixing, curing, or other designated applications shall be fresh water, clean and free from oil, salt acid, alkali, sugar, vegetable or any other substance injurious to the finished product. The water shall meet the requirements of the Standards, in particular DIN 4030 or BS 3148.

The Contractor as per BS 3148 shall examine water obtained from a source other than a public piped supply of potable water with regard to its suitability for producing concrete. The certificates are to be submitted to the Employer.

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Nominal Concrete Mix

Concrete for concrete blocks shall correspond to concrete Class B 25, DIN 1045, with the following requirements (tested at cube specimen of size 20 x 20 x 20 cm):

- minimum compressive strength after 28 days of each test cube: 25 N/mm²
- minimum average compressive strength after 28 days of each series of three test cubes: 30 N/mm².

The minimum cement content shall be 350 kg for one m^3 of ready mixed concrete. The free water to cement ratio shall be within 0.50 to 0.60 by weight.

The Contractor shall determine the optimal mix proportions through preliminary tests in consideration of the site conditions, the intended transportation and placing method and as otherwise required by the respective standard (DIN 1045).

Quality Control

During production of concrete blocks quality tests shall be performed in accordance with DIN 1045 or BS.

Six test cubes of 20 x 20 x 20 cm shall be prepared on every day of block production for each 100 m^3 of concrete poured to verify the compressive strength. In case more then one batching plant is supplying material for CC-block production, one set of six test specimen is to be produced for each plant on every working day. Alternatively cylindrical test specimen may be used.

Three each of each set of the test specimen shall be tested after 7 days and 28 days of its production. The compressive strength shall at least correspond to the minimum values stipulated above.

Production

Formwork and moulds shall ensure maintaining designed shapes and block sizes. They shall preferably of steel.

Mixing of concrete shall be done thoroughly to ensure that concrete of uniform color and consistency is obtained. Unless otherwise permitted by the Employer hand mixing of concrete is prohibited. Batching plants shall be used for mixing concrete.

The ingredients of concrete such as cement, fine aggregates, coarse aggregates and water shall be measured correctly for each batch of mixing. In case of volumetric batching the bulking of aggregates must be accounted.

Concrete shall be transported from the place of mixing to the place of final deposition as quickly as possible. The methods adopted should ensure that concrete is placed in position within 45 minutes, so that the evaporation and segregation of the mix is prevented. Rehandling shall not occur.

Concrete shall be placed directly in its final position avoiding segregation. Concrete should be placed gently at its position and not thrown from a height. Before placing concrete the formwork and moulds shall be cleaned and well wetted, to the satisfaction of the Employer.

Compaction of concrete shall be properly done to secure maximum density and strength. It shall be done immediately after placing of concrete.

Dependent on the size of blocks to be produced compaction may be done by hand (rodding and tamping) or by mechanical vibrations, subject to Employer's approval.

Curing shall begin as soon as the concrete is sufficiently hard and shall be continued for 10 days. Curing methods may be by spraying water to the concrete, or by covering the concrete surface with a layer of gunny bags, canvas, hessian, straw or similar absorbent materials which is to be kept constantly wet, or another method approved by the Employer.

Side formwork may be removed after one day, if approved by the Employer.

The ready concrete blocks shall not deviate by more than ± 5% by weight from the size specified on the Drawings or in the Bill of Quantities.

Concreting at High Ambient Temperatures

All measures required by the Contractor for carrying out concreting at high ambient temperatures should be coordinated with and be acceptable to the Employer.

The temperature of fresh concrete shall not exceed 35°C, when placed in the forms. As the case may be concreting shall be carried out early in the morning or at night under exceptionally hot weather conditions.

If the Contractor intends to use special retarding or liquefying additives for the concrete mixes preliminary concrete tests must be carried out at the expected placing temperature, to determine and verify the effects of such concrete additives.

Immediately after its compaction, the concrete must be provided with an effective protection against desiccation. Otherwise, high temperatures in combination with strong air flow and low humidity may adversely affect the quality and soundness of the respective structural concrete member.

Desiccation of the water contained in the concrete may be prevented by immediate covering the fresh concrete with water-proof and vapor-proof foils or films, or by other appropriate methods to be proposed by the Contractor and acceptable to the Employer

Steel Wire Cables

For standardized structures the steel wire cables used in the ACM shall at least fulfill the following characteristics:

Structure Category	**SC 2**	**SC 3**
Diameter (minimum):	10 mm	15 mm
Number of wires:	7 wires 	19 wires 
Tensile strength (min.):	≥ 1,570 N/mm²	≥ 1,570 N/mm²
Rupture strength (min.):	≥ 1,300 N/mm²	≥ 1,300 N/mm²
Rupture load (min.)	80 kN	180 kN
Elongation (max.)	0.75 %	0.75 %
Number of cable clamps at any cable overlap or joint	> 3	> 4

Table 7.10-1: Grading curves of concrete aggregates for ACM

7.10.2 Articulating RENO® Mattress System

7.10.2.1 General Description

The main advantages of an integrated mattress system are:

- articulating behavior
- enhanced flexibility and stability over individual protective elements
- reduced risk of localized failure
- ease of construction as done in dry construction pit
- minimized problems regarding vandalism and theft

Single RENO® mattress units can be assembled to an integrated mattress system providing such an improved sustainability against erosion processes. To ensure integrity of the system, parallel and perpendicular steel wire cables are run through the mattress units. All cables running perpendicular to the embankment slope are to be anchored to deadmen at the base of the upper revetment.

The block size is to be determined using the formulas of Chapter 5 and Design Plate E for current attack and wave attack respectively. The cabling should not be taken into account for determining the individual block sizes, but they are provided to enhance the system stability in case of extreme conditions from non-anticipated hydraulic forces or excessive riverbed erosion processes.

A suitable geotextile filter mat should be provided below the mattress, at least conforming to the criteria stipulated under Section 7.4.2.3.

Figure 7.10-7: Overall view of completed launching apron with articulating RENO®-mattresses

The cables should be multi-core steel, but not from synthetic fibers. The latter are plastic based, and their durability as well as excessive elongation under load rule out their use. Furthermore, they are more vulnerable to abrasion and vandalism.

Generally, RENO®-mattresses would suit installation below water as mattress units may be pre-assembled to larger. For site handling, transport and placing suitable steel frame arrangements are to be provided, to which the ready-made mattress can be suspended. Heavy crawler cranes with large out-reach are required, on-shore as well as off-shore. The flow conditions of the major rivers of Bangladesh would substantially increase the construction risks; even prevent the execution at times. Therefore, the use of RENO®-mattresses for under-water installation is not recommended for Structure Categories SC 2 and SC 3.

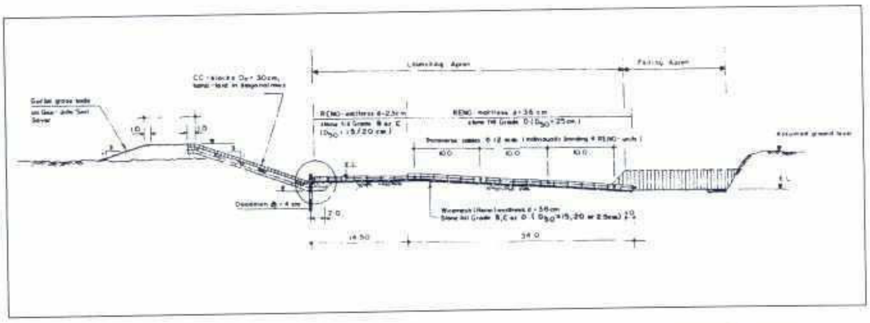

Figure 7.10-8: Typical cross-section of launching apron with articulating RENO®-mattress

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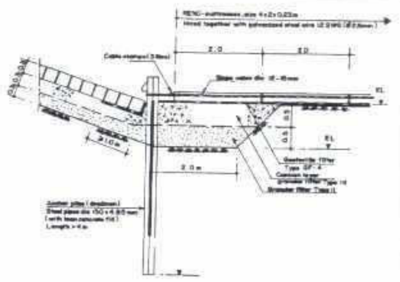
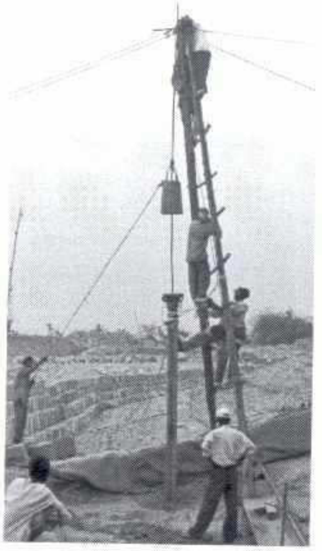
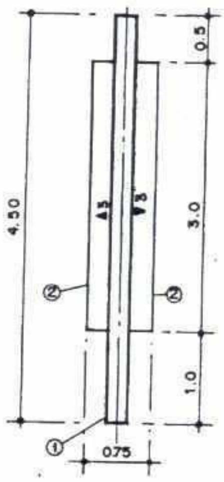


Mattress anchorage at the base of upper revet-ment (berm level)



Installation of deadmen for mattress anchorage by guided drop weight



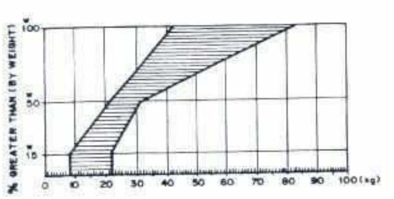
Typical deadman design (steel tube ø 169 x 5 mm)

Installation stage of deadmen along toe of upper embankment (berm level)

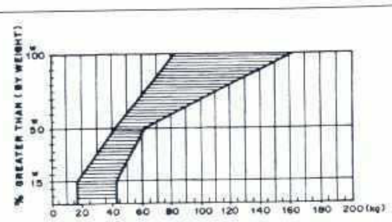
Initial stone filling of RENO®-mattresses (prior to installing steel wire cables)

View along deadmen anchorage along toe of upper embankment (berm level)

Figure 7.10-9: Details of RENO ® mattress systems



Stone fill – Grading Range C – D_{50} = 20 cm
(for 23 cm thick RENO®-mattresses)



Stone fill – Grading Range D – D_{50} = 25 cm
(for 36 cm thick RENO®-mattresses)

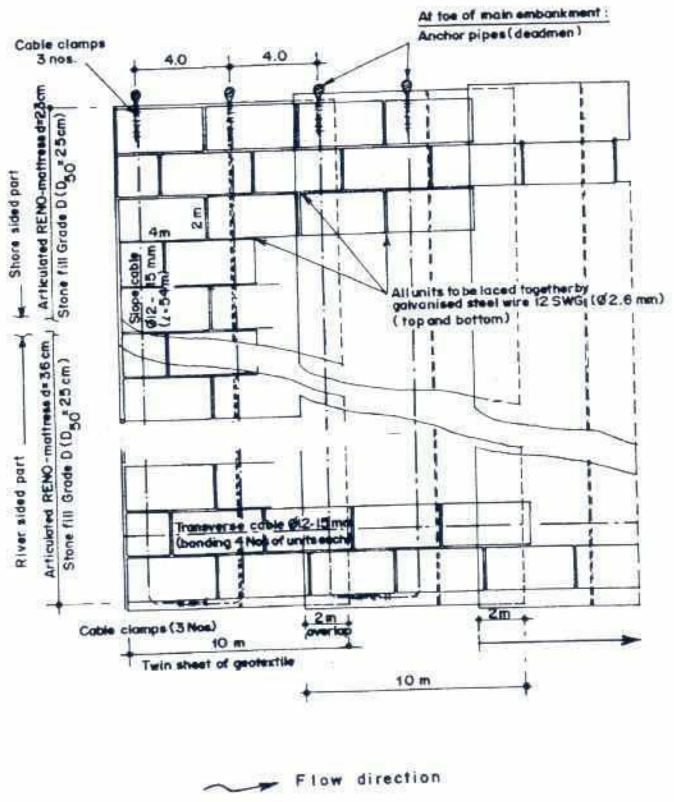


Figure 7.10-10: General layout of articulating RENO®-mattress for Structure Category SC 2

7.10.2.2 Material Specifications

(a) RENO® Mattress Units

RENO®-mattresses are large and thin box-type construction elements made of zinc plus PVC-coated hexagonal double twisted wire mesh (wire diameter 2 mm / 3 mm including coating layers).

Single mattress dimensions of 4 x 2 m are recommended; with partitions at every 1 m. Recommended mesh size about 60 x 80 mm.

The mattress thickness [d] depends on the type of fill material and the hydraulic design loads. Stone or rock fill of the following size is recommended.

- d = 23 cm (Structure Category SC 2, and over the width of the berm);
- d = 36 cm (Structure Category SC 3, and exposed zones within SC 2).

The wire mesh must comply with the following factory specifications:

- Tensile strength: both, the wire used for the manufacture of mattress and the lacing wire, must have a tensile strength of at least 38-50 kg/mm^2 according to BS 1052/80 "Mild Steel Wire". These values are referred to original wire before manufacturing the mesh.
- Zinc coating at 240 g/m^2 meeting the requirements of BS 443/32 or DIN 1548.
- PVC-coating must have a thickness of at least 0.5 mm, conforming to ASTM.

(b) Stone Fill

General

Within this Section "stone" shall include natural stone material obtained from rivers or quarries.

The material supplier shall satisfy himself that the sources of materials in Bangladesh or abroad have sufficient capacity to meet the requirements of the project. In the context of capacity, both the quantity of production and its compliance with the delivery schedule must be taken into account.

Material Standard and Tests

The material shall be free from dirt, sand, gravel and materials of organic origin (roots etc.), and shall be good sound stone / rock without cracks/fissures to avoid breaking during handling and placing or dumping.

The following standards shall apply:

- BS 812: Testing aggregates
- ASTM C 88: Aggregate soundness
- ASTM C 535: Abrasion resistance

227

The following test result shall be verified and respective reports and certificates submitted:

- The weight average loss shall not exceed 10% by weight, when subjected to the ASTM C-88 Sodium Sulphate Soundness Test
- The average bulk specific gravity of any sample shall be in the range of 2600 kg/m3 (BS 812, Part 2, Chapter 6, 3)
- Water absorption of rock material shall not exceed 6% (BS 812, Part 2, Chapter 5)
- Minimum compressive strength as per ASTM C 170-50: 100 N/mm²
- The percentage of wear shall no exceed 40% (ASTM C 535), when subjected to the Los Angeles Test

Only stone / rock with a factor not exceeding 2.5 between the longest and shortest dimension of the rock shall be allowed in the delivery.

Grading Tests at Point of Delivery

Testing at the agreed point of delivery (stockpile yard) shall take place at the supplier's expenses. The location for testing shall be arranged by the Contractor and approved by the Employer.

The gradation of materials stock piled at the yard shall be tested at least one time for each 500 t of delivery.

Samples for determination of weight gradation shall contain at least 100 individual stones / rocks. The samples shall be taken by random selection from each specified gradation to obtain representative samples, and shall confirm to the range recommended in the following tables.

Should tests show non-compliance with the specified gradation range the Employer may order more tests to be performed. In case the additional tests show non-compliance the Employer may reject whole or part of the rock delivery.

Rejected material shall be removed from the stock pile yard or blended with suitable rock sizes by the Contractor at his own expense, all to the approval of the Employer.

Range C	% Smaller than (by weight) [kg]			Stone Sizes [cm]		
	W_{15}	W_{50}	W_{100}	D_{15}	D_{50}	D_{100}
min.	8.3	20.8	41.6	15	20	25
max.	21.8	31.2	83.2	20	23	32

Table 7.10-2: Fill for RENO®-mattresses of d = 23 cm

Range D	% Smaller than (by weight) [kg]			Stone Sizes [cm]		
	W_{15}	W_{50}	W_{100}	D_{15}	D_{50}	D_{100}
min.	16.3	40.6	81.3	18	25	32
max.	42.7	60.9	162.5	25	29	40

Table 7.10-3: Fill for RENO®-mattresses of d = 36 cm

In the above tables the following definition applies:

D_{50} = nominal size/diameter of stones

W_{15} = 15% of the stones of a gradation range may be less than the weight specified against W_{15};

W_{50} = 50% of the stones of a gradation range shall be at least of the weight specified against W_{50};

W_{100} = 100% of the stones of a gradation range shall not exceed the weight specified against W_{100}.

7.11 FALLING APRONS

7.11.1 General Description

"Falling apron" is a multi layer system placed on a sloping or horizontal surface as scouring protection. The individual units articulate freely following the morphodynamic forces of the river and stabilize the eroding bank. The single weight of each unit and the volume of protective material within a defined area are the decisive factors for designing an efficient falling apron.

The size and volume of material may be determined using the formulas of Chapter 5 and the Design Plate F.

The volume of falling apron material when placed in dry condition, i.e. on an excavated surface just above Standard Low Water, as recommended for the standardized structures SC 2 and SC 3, can be well controlled by counting the number per area.

Below the water level falling apron material shall be laid by controlled dumping considering a nominal voids ratio of 35%.

The nominal thickness of a falling apron layer shall be achieved over at least 75% of the area and nowhere shall the coverage be less than 80% of that as per Specification.

CC-blocks have proven to be very effective, but more expensive then boulders, rock or geo-sand containers.

For standardized structures concrete blocks are recommended, but geo-sand containers may also be used, especially for structures with a limited design life.

7.11.2 CC-Blocks

(a) Material

Production of cement concrete blocks shall follow the general descriptions and Specifications presented under Section 7.5 (cubical concrete blocks).

(b) Placing Volume of CC Blocks

For falling aprons of cc-blocks the quantity of material to be provided within the designed falling apron width W_{fa} is defined by the number of blocks to be placed within an area of 100 m². This will facilitate proper control of material placement.

The figures presented in the table below serve as a guideline for standardized structures, but the correct size and volume should be determined by using the Design Plates.

CC-Block Size D_n [cm]	25		30		35		40		45	
	d_{fa} [m]	Nos [-]	d_{fa} [m]	Nos [-]	d_{fa} [m]	Nos [-]	d_{fa} [m]	Nos [-]	d_{fa} [m]	Nos [-]
Inner part (1/3 W_{fa})	0.40	2,100	0.45	1,300	0.55	1,000	0.60	720	0.70	600
Outer part (2/3 W_{fa})	0.85	4,100	1.00	2,800	1.20	2,100	1.35	1,600	1.50	1,250
Exposed edges	0.85	4,100	1.00	2,800	1.20	2,100	1.35	1,600	1.50	1,250

Table 7.11-1: Number of cc-blocks per 100 m² of falling apron surface area

7.11.3 Geotextile Sand-Container

(a) Material

Geotextile filter material shall be used for tailoring geo-sand containers to be used in falling aprons. General descriptions are presented under "Revetment Works, Geotextile Filter Materials".

The type (quality) of geotextile to be selected depends on the scheduled filling volume of a single unit. The recommended minimum criteria are presented in a table attached hereto.

(b) Dimensioning

Contrary to blocks or boulders, where only the ground area is in contact with the river bed and the scour slope respectively, sand-filled geo-container cover a considerably larger area with identical unit weight, irrespective of their changing shape. Thus, they represent a relatively thin, but area-wise large protection element.

It is assumed that a theoretical coverage of the anticipated scour slope of at least two layers of flat-laying sand-containers should be adequate to stabilize the eroding bank. The recommended number of geo-sand containers to be placed within 100 m² of falling apron area within a standardized structure are compiled in the table below, but the correct size and volume should be determined analogously to cc-blocks.

For some typical cc-block sizes, the equivalent size of sand-filled geotextile containers is given in Table 7.11-2.

Geo-Sand Container Type/Volume	B	C	D	E
	125 kg/No	180 kg/No	250 kg/No	900 kg/No
Equivalent CC-block D_n [cm]	40	45	50	-
Inner part (1/3 W_{fa})	420	320	260	120
Outer part (2/3 W_{fa})	690	520	430	200
Exposed edges	-	520	430	200

Table 7.11-2: Number of geo-sand containers per 100 m² of falling apron surface area

Casting of CC-blocks directly in-situ at the designated falling apron area of a structure.

(This alternative to the favoured dumping of cc-blocks is cost effective, but articulating behaviour of the mass of cc-blocks is less effective, thus not recommended)

Falling apron of CC-blocks exposed to the river

Falling apron of geo-sand containers during construction stage

Dumping of CC-blocks within the designated falling apron area of a structure

Filling of geo-sand containers at site (directly at the designated location within the structure)

Covering of the completed launching and falling aprons as a protective measure against human intervention

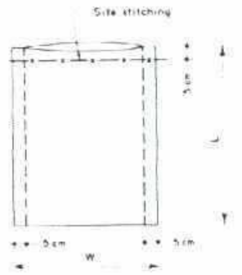
In-situ filling and placing of geo-sand containers within the designated area

Figure 7.11-1: Placing of CC-blocks and geo-sand containers

202

(c) Tailoring

Giving consideration to the fact that a single protection unit shall practically not move under the hydraulic forces and wave loads, the equivalent weight of one protection unit can roughly be determined by using Design Plate A.



Typical stitching plan for a geo-sand container

Concrete Block Size D_n [cm]	40	45	50	75
Geo-sand Container Type	**B**	**C**	**D**	**E**
Equivalent weight [kg]	125	180	250	900
Volume (~ 80% fill) [m^3]	0.07	0.10	0.14	0.50
Geotextile size of container L x W [m]	0.96 x 0.88	1.20 x 0.88	1.60 x 0.85	2.15 x 1.27
Theoretical area covered by 1 container [m^2] [1)]	0.60	0.75	1.00	2.0
Theoretical thickness d_n [m]	0.12	0.13	0.14	0.25

[1)] net, excluding material loss for stitching (2 x 5 cm) and 15% shrink due to filling

Figure 7.11-2: Tailoring details for geo-sand containers

(d) General Specification

Geotextile materials shall generally be stored under cover, well sheltered from direct sunlight and to prevent the ingress of dust or mud. They shall be protected from damage by insects or rodents.

Geotextile containers shall be fabricated from geotextile material complying with the following specification:
Each container shall be stitched along all cut edges, except for the opening at the top of each container, which shall be wide enough to allow the filling. Container Type A may be manufactured with double locked stitching, but for Types B to E special oversew seam shall be applied.

Thread shall be PES white and PES black, which should be supplied from the manufacturer of the respective geotextile material.

The minimum tensile strength of any seam shall be not less than 90% of the tensile strength of the geotextile itself.

Geotextile bags shall only be filled to 70 to 80% of the theoretical bag volume.

The fill material may be obtained from excavated or dredged material interim stored at the Site. The amount of silt contained in the fill should be kept as low as possible.

The fill opening of the containers shall be closed tightly after filling and then closed with double lock chain stitch, using hand-held sewing machines.

Description	Unit	Container Type	
		B - C - D	E
Effective opening size O_{90}	mm	< 0.09	< 0.09
Area mass	g/m²	≥ 400	≥ 500
Thickness	mm	≥ 3.5	≥ 3.5
Min. tensile strength as per DIN 53857			
- longitudinal	N/10 cm	≥ 1,700	≥ 2,200
- transversal	N/10 cm	≥ 2,700	≥ 2,700
Min. elongation			
- longitudinal	%	≥ 50	≥ 50
- transversal	%	≥ 50	≥ 50
Coefficient of permeability at a load of 2 kN/m²	m/s	$> 1x10^{-7}$	$> 1x10^{-7}$
Materials		PP/PES	PP/PES

Table 7.11-3: Specifications of geotextile material for manufacture of geo-sand containers

8.1 PILE INSTALLATION PROCEDURE AND EQUIPMENT

8.1.1 Pile Categories

For the application in standardized bank protection works and erosion prevention measures within Bangladesh the envisaged standard materials are compiled in Table 8.1-1. However, the table shall not be a restriction for other materials or solutions.

Pile Category	Material	Diameter / Size	Max. Length
Timber Piles	Bamboo bundles	< 150 mm	< 7.5 m
	Bullah piles	< 200 mm	< 8 m
Concrete Piles	Precast reinforced concrete	500 x 500 mm	15 m *(cast at site)*
	Prestressed spun concrete	Ø 500 mm	3 @ 10 m = 30 m
Tubular Steel Piles	Mild steel (or higher)	Ø < 1,500 mm	No limitation
Sheet Piles	Precast reinforced concrete	250 x 500 mm	< 10 m
(for cofferdams only)	Special sheet pile steel	Factory standard	No limitation

Table 8.1-1: Pile categories

The limitations presented in Table 8.1-1 are given:

- for timber/bamboo piles by nature;
- for prestressed spun concrete piles by the facilities of the only Bangladeshi manufacturer (diameter 500 mm) and length by road transport restrictions within Bangladesh (presently 10 m)
- for precast reinforced concrete pile by site handling and available length of pile installation leader
- for factory-made precast reinforced concrete sheet piles by road transport restrictions within Bangladesh (presently 10 m)
- for tubular steel piles by manufacturing facilities within Bangladesh (presently max. diameter 4 feet = 1,220 mm)

8.1.2 Categories of Pile Driving

This Section presents some criteria for deciding the appropriate pile installation equipment. Thereby, the definition of the term "installation" in this Section means the sinking of piles or sheet-piles to their design depth by appropriate methods in consideration of the prevailing local and subsoil conditions and of the Specifications. This is usually done by either pile driving hammer or vibrator or a combination of both methods. Other methods, such as pressing or installation into an excavated trench, are only used in exceptional cases which do not apply for the erosion protection measures considered herein.

2ⁿ7

The selection of piling equipment depends mainly on:

- Subsoil conditions
- Pile material
- Pile dimension (by diameter, length as well as weight)
- Location, i.e. onshore or offshore piling

Pile driving hammers may be simple drop hammers, steam or air operated hammers, diesel hammers or hydraulic hammers, while vibrator hammers may be the electric or hydraulic driven type. Under normal circumstances the selection of the most suitable pile installation method is decided by the type of subsoil prevailing at the location and the size, weight and material of the pile to be installed. However, there are situations that the actually available pile installation equipment presents a constraint for the design engineer, who may be required in such a case to adopt the pile design and dimensions to suit the only available equipment.

The recommendations for pile installation equipment presented in the following subsections are to be interpreted within and in consideration of the above limitations. It has further been considered that the subsoil to be penetrated during pile installation does normally not present a constraint for these works.

8.1.3 Pile Installation by Vibration

Pile installation by "vibration pile hammers" is an easy and fast way of pile driving. Vibration frequencies, weight and size of the vibrator hammer must be chosen according to the afore-mentioned conditions. An advantage of vibrator pile hammer is that they can easily be used also for extracting piles in case of misdriving, encountering of an obstacle in the ground, etc. However, pile driving by vibration has its limit when the pile driving force is excessive due to subsoil response and consequently the depth of penetration over time reduces against zero.

Alteration of the vibration frequency can support to reach greater installation depths. But most vibrator hammers that are available in the market are provided with single frequency only, wherefore a second or even a selection of different vibrator hammers may have to be maintained at a site if subsoil conditions call for such measure. Vibrator hammers with multiple frequencies are available from reputed manufacturers, but very expensive and possible beyond the financial feasibility of the projects in question.

Vibrator pile hammers are powered in general either directly by an electric motor or by hydraulic motor. They are mostly used without a leader and lifted by cranes to the top of the pile. However, to install the pile and to keep its position a "pile frame" (which can also be a leader) is suggested.

Pile installation by vibration is not recommended for concrete piles because the vibration may destroy the concrete.

While piling hammers can be utilized for practically all subsoil situations, vibration processes are somewhat restricted and less efficient if not unsuitable at all for densely deposited granular soils, hard cohesive soils, particularly those with a low water content.

Exemplary and as a very rough guide the order-of-magnitude of a vibrator capacity for steel sheet pile installation is given in Table 8.1-2.

SPT N-Value	Type of Soil	Minimum Centrifugal Force per Linear Metre of Steel Sheet Pile
	Non-Cohesive Soil	
0 - 4	Very loose	10 kN
4 - 10	Loose	15 kN
10 - 30	Medium dense	20 kN
30 - 50	Dense	25 kN
> 50	Very dense	40 kN
	Cohesive Soil	
0 - 2	Very soft	10 kN
2 - 4	Soft	15 kN
4 - 8	Medium hard	20 kN
8 - 15	Hard	25 kN
15 - 30	Very hard	40 kN
> 30	Extremely hard	50 kN

Pile installation by vibrator (typical arrangement)

Table 8.1-2: Guide values for determination of vibrator capacity for normal steel sheet piles

Within the Pilot Project FAP 21 pile installation by vibration was executed at "Test Site Kamarjani", for the part of permeable groyne structure with tubular steel piles. All steel sheet piles and steel piles of diameter 711 mm, 1016 mm and 1220 were installed by vibration. The piles of diameter 711 mm could be installed to their final design depth of more than 20 m by vibration only (total pile length 28 m). Piles of diameter 1016 mm and 1220 mm were installed up to the maximum possible penetration by vibration (varying between 18 to 24 m). The final penetration depth up to over 30 m (total pile length 42 m) was reached by subsequent piling with a hydraulic drop weight hammer.

8.1.4 Pile Installation by Hammer

The most common equipment for pile driving are simple "single drop hammers", "diesel hammers" and "hydraulic drop weight hammers". These are available in various sizes in Bangladesh. During the execution period of the FAP 21 Pilot Project the following piling hammers were available within Bangladesh:

- single drop hammers up to 5 t (but any weight could be manufactured, always provided that the suitable pile installation tripod and winch would be made available);

- diesel hammers from 0.5 t up to 3.3 t stroke weight, with a corresponding piling energy of about 12.5 kNm up to about 90 kNm, and
- hydraulic piling hammers of 3.5 t, 7 t and 15 t with a corresponding piling energy of 42 kNm up to about 150 kNm.

Weight of Stroke	Piling Energy per Blow	No. of Blows per Minute	Suitable for Pile Weight up to
5 kN	12 kNm	40 – 60	1.5 t
12 kN	31 kNm	40 – 60	4 t
22 kN	34 – 76 kNm	38 – 52	6 t
30 kN	44 – 87 kNm	38 – 52	8 t
36 kN	52 – 115 kNm	37 – 53	10 t
46 kN	67 – 146 kNm	37 – 53	15 t
55 kN	86 – 160 kNm	36 - 47	20 t

Pile installation by hydraulic hammer
(typical arrangement)

Table 8.1-3: Typical diesel hammer data

Typical piling hammers are mounted to so-called "leaders" attached to a support vehicle, such as crawler crane or similar. This typical arrangement ensures the exact pile direction (vertical or inclined) as well as centrally positioning of the pile hammer. Contrary to this fixed arrangement, there are "flying leaders" (mostly for installation of inclined piles) and self-riding piling hammers (mostly for installation of vertical piles), which are suspended by ropes to the support vehicle (crawler crane). The advantage is a higher flexibility and mobility at site, in particular when somewhat remote pile locations must be reached within the structure layout, or when working offshore from a barge pontoon.

Very important for successful and efficient pile installation by driving hammers is the right choice of the pile cap arrangement. Commonly a cushion is being provided between the hammer's anvil and the pile cap. Dependent of the type of pile, the prevailing subsoil (i.e. the subsoil response to be expected during penetration of the pile into the ground) and the available hammer capacity this cushion can compensate between extremes, i.e. control the effective driving force introduced into the pile by the hammer. Figure 8.1-1 presents some typical pile head packing and the respective energy reduction factors η.

8.1.5 Installation of Timber Piles

Apart from the aspect that use of timber piles should be waived in line with nowadays environmental understanding and only bundles of bamboo poles be utilized instead for short-term measures, these can easily be installed with simple drop weight. For simple situations a tripod with SPT (Standard Penetration Test) drop weight will suffice, guided at the pile's head by a center bar, while manpower or simple diesel-driven winches are lifting the weight. This method can easily be adopted for drop weight up to 150 kg, which suffice to drive such piles (or piles of other materials but with similar dimensions) up to about 8 m into the ground. The method can be improved by employment of a diesel driven winch for lifting the heavy drop weight.

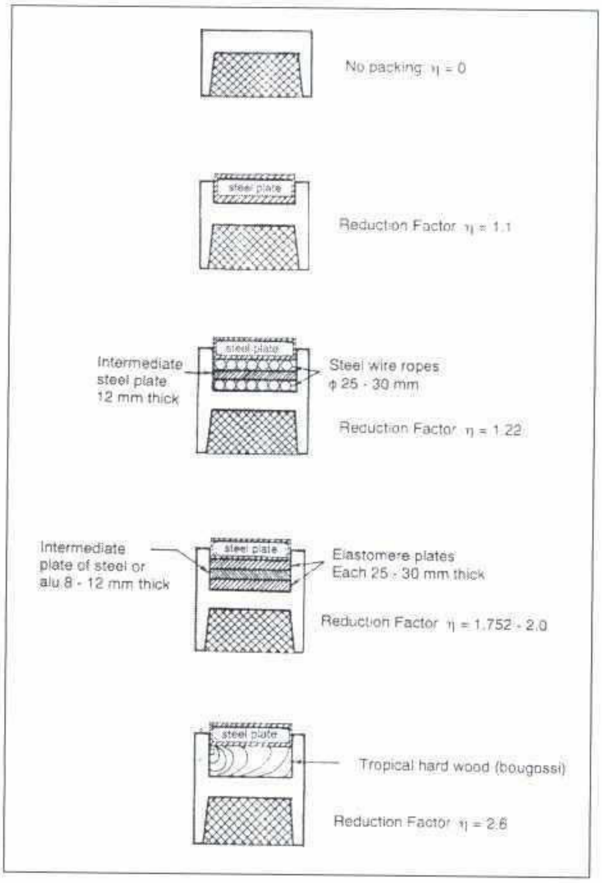


Figure 8.1-1: Typical pile cap packing and related reduction factors η

8.1.6 Installation of Pre-Cast Reinforced Concrete Piles

The term "precast reinforced concrete pile" within the context of this Section represents any type and shape of its kind, e.g.

- Factory-made pre-cast or prestressed spun concrete piles (which usually are of higher grade and quality);
- Site-manufactured reinforced concrete piles, and
- Factory-made reinforced concrete sheet piles.

It is common for any pre-cast concrete pile that its installation by driving must be carried out such that the concrete does not suffer beyond tolerable limits. This is best achieved when driving is being carried out with a heavy drop weight but at little drop height.

Therefore, hydraulic piling hammers are perfect for this purpose, since the drop height can be pre-set and controlled throughout the installation process. The adjustable drop height matches perfectly the material characteristic of concrete piles and reduces the risk of pile damages during installation considerably. Initially, the drop height should be controlled at 20 cm and successively be increased, preferably up to about 0.5 m only, but it should not exceed 1 m. The hammer stroke should be regular and may not exceed 40 blows per minute.

As an added advantage, hydraulic piling hammers are environmentally more acceptable then diesel piling hammers, unless these are provided with anti noise and pollution cage.

The old rule that the weight of the hammer should be at least equal but preferably heavier than the pile itself applies much for concrete piles still today although in many cases this can not be adhered to any more. The ratio pile weight (W_P) to drop weight (W_D) should, however, be maintained in the range of 1: 0.7 up to 1: 1.5. For the size and weight of pre-cast reinforced concrete piles recommended for standardized structures and the anticipated moderate subsoil conditions a hydraulic hammer with a drop weight of 6 t to 8 t will usually suffice.

With moderate, i.e. not too dense soil conditions concrete piles up to size/diameter 500 mm and length of 20 m may also be driven by middle size diesel hammer.

8.1.7 Installation of Steel Piles

Within the context of this Section the term "steel pile" represents any type and shape of its kind, e.g.

- Steel sheet pile (factory made and imported);
- Tubular steel piles of various diameters and material grades, and
- Rolled steel sections, such as H-type and hollow box-type.

For the installation of steel piles of the type recommended for standardized structures, i.e. of diameter up to 1220 mm and a pile length of up to about 45 m (pile penetration into the subsoil of about 30 m) a hydraulic hammer with a drop weight of 15 tons and maximum height of drop of 1 m is a recommended solution. At the Jamuna Multipurpose Bridge steel piles of 68 m to 86 m length and diameter up to 3050 mm were driven with a hydraulic hammer with a drop weight of 150 tons. This hammer was specially designed for that project.

As compared to solid concrete piles most steel piles, and in particular hollow steel pile sections are less critical for proper installation, since obstacles encountered during installation, including soil layers that are difficult to penetrate, can be handled. On the other hand pile heads or even the entire pile can suffer damage if the steel section has not been well designed in consideration of the driving resistance to be expected during the installation process. As a rule of thumb the minimum steel pile wall thickness should not be less than $1/100^{th}$ of the pile diameter, i.e. for a pile diameter of say 1,200 mm the minimum wall thickness should be 12 mm at the pile tip and the pile top unless for statically design reasons a thicker steel pile wall is needed.

Therefore, as in any other case, the decision on the suitable pile installation method and equipment has to be made in advance, well in consideration of the prevailing situation and the type and quality of pile material available.

In recent years it has become common to install piles by vibration method. This is particularly advantageous if piles are to be installed without a leader that is the guide, which ensures that the pile as well as the hammer is well and centrally supported during the installation process.

However, whenever utilising vibrators for pile installation one must be aware of the intended purpose and functioning of the pile. Those, which are structural elements to carry vertical loads (e.g. those within foundations for bridges, buildings, etc.), have to be driven by a hammer for at least the last 3 m to 5 m before reaching the design depth, to ensure activation and improvement of the point bearing of the pile. For the piles within a permeable groyne structure, however, this is not relevant and piles may be installed entirely by vibration. However, in practical terms the vibrator capacity should not be over-sized for economical and handling reasons, but a suitable hydraulic piling hammer should supplement the pile installation equipment. With such a set-up also unforeseen, more difficult subsoil conditions can be tackled.

Section 8.1.8 provides some basic elements in order to plan the sufficient pile installation equipment. Again, these are suggestions from engineering judgement, but the subsoil conditions at a given site and the pile size and type to be installed must always be given due consideration in the thoughts of the planning engineer.

8.1.8 Estimation of Piling Hammer Capacity

8.1.8.1 Estimation of Driveability, Skin Friction, Pile Tip Resistance and Pile Driving Resistance

Prior to any pile driving works the condition of the subsoil should, among others, be checked by Standard Penetration Tests (SPT) or Cone Penetrometer Tests (CPT). Depending on the determined CPT or SPT-values the characteristic of non-cohesive and cohesive soils can be tentatively assumed as indicated in Table 8.1-4 and Table 8.1-5.

CPT: q_c [MN/m²]	SPT – value [-]	Characteristic Description
< 2.5	< 4	very loose
2.5 – 7.5	4 – 10	loose
> 7.5 – 15.0	10 – 30	medium
> 15.0 – 25.0	30 – 50	dense
> 25.0	> 50	very dense

Table 8.1-4: Characteristics of non-cohesive soils

CPT: qc [MN/m²]	SPT-value [-]	Characteristic Description
< 0.25	< 2	very soft
0.25 – 0.50	2 – 4	soft
0.5 – 1.0	4 – 8	medium
1.0 – 2.0	8 – 15	stiff
2.0 – 4.0	15 – 30	very stiff
> 4.0	> 30	hard

Table 8.1-5: Characteristics of cohesive soils

With the chosen pile type and the classified subsoil, skin friction (τ_{mf}) and pile tip resistance (σ_{sf}) for static loading can be estimated using the general assumptions compiled in Table 8.1-6: . Thereby it should be noted that these figures are only presented for determination of a piling hammer capacity, but not for the purpose of structural or otherwise design assumptions.

Driveability	Subsoil	τ_{mf} (kN/m²)		σ_{sf} (MN/m²)	
		Reinforced Concrete Piles	Steel Piles	Reinforced Concrete Piles	Steel Piles
very easy	cohesive, very soft to soft	5	5	0.25	0.25
easy	medium and coarse sand, loose	60	50	4.00	3.50
	cohesive, soft	15	15	0.50	0.50
medium	medium and coarse sand, medium	60	60	5.50	5.00
	cohesive, medium	20	20	0.75	0.75
difficult	fine sand	60	70	7.00	6.50
	cohesive, stiff to very stiff	30	30	1.50	1.50
very difficult	fine sandy and silty, dense	60	75	8.00	7.50
	cohesive, hard	45	45	2.00	2.00

Table 8.1-6: Subsoil characteristics versus driveability of subsoil

The pile driving resistance W_e that may have to be expected during the final pile installation process (i.e. before reaching the designed penetration depth of the pile) can be calculated according to the following formula:

Estimated pile driving resistance: $\boxed{W_e = U \cdot l_e \cdot \tau_{mf} + A_t \cdot \sigma_{sf}}$ [kN] (Eq. 8.1-1)

For explanation of symbols see Section 8.1.8.7.

8.1.8.2 Determination of Minimum Wall Thickness, Material Properties, Weight of Pile and Maximum Permissible Driving Force

In order to drive a pile safely into the ground the pile characteristics (material and thickness) must, apart from otherwise relevant design aspects, be chosen to withstand the expected driving resistance. Consequently, it may be assumed that a pile can be driven into the subsoil undamaged (provided always that there is no obstacle in the subsoil) when the permissible driving force F for the chosen pile is at least equal, but normally higher than the estimated pile driving resistance W_e, i.e. $F \geq W_e$.

For steel piles the permissible driving force F depends on the steel cross-sectional area and the yield stress f_y of the material:

Permissible pile driving force: $\boxed{F = U \cdot t_{\min} \cdot f_y}$ [kN] (Eq. 8.1.2)

For explanation of symbols see Section 8.1.8.7.

Using formula (Eq. 8.1-3) the required minimum wall thickness of a steel pile may be computed. As the case may be, this t_{min} may be higher, i.e. thicker, then the steel wall thickness required only for the horizontal and vertical loading as per design calculations. In such a case the pile sections at the head and pile tip should be strengthened to meet with the stresses expected during the pile installation process.

Required wall thickness of steel piles: $\boxed{t_{\min} \geq \frac{W_e}{(U \cdot f_y)}}$ [kN] (Eq. 8.1-3)

The finally chosen wall thickness $t_e > t_{min}$ depends on the otherwise structure loads and design requirements, the choice of standard/available pipe sections, respectively the pile manufacturers facilities.

The maximum permissible driving force is calculated using t_e:

Maximum permissible driving force: $\boxed{F_{\max} = U \cdot t_e \cdot f_y}$ [kN] (Eq. 8.1-4)

8.1.8.3 Choice of Hammer Type and Initial Determination of Hammer Weight

For the choice of a suitable pile driving hammer the following rule of thumb may be used as a first approximation.

Type of Hammer	Steam Hammers Pressurized Air Hammers Hydraulic Hammers Drop Hammers	Diesel Hammers (with hit dispersion)	Diesel Hammers (with high pressure injection)	Hydraulic Hammers (with adjustable striking force)
Proportion of pile weight (incl. cap) to weight of hammer	1 / 1 to 1.5 / 1	2 / 1 to 3 / 1	1 / 1 to 1.5 / 1	1 / 1 to 2 / 1

Table 8.1-7: Initial approximation of piling hammer type and weight

The weight of appropriate pile caps depends on the type and the size of the pile itself and the hammer. They usually range between 700 kg and 3000 kg.

8.1.8.4 Typical Hammer Data and Pile Cap Factors

Table 8.1-8 and Table 8.1-9 present typical data of two manufacturers of piling hammers. The respective data may be applied for tentative determination of the piling hammer through application in the formulae presented in the following Sub-Chapter. However, for more correct application only the latest data of reputable piling hammer manufacturers should be used.

Type	Weight of Ram	Weight of Pile Cap	Min. Drop Height min H	Max. Drop Height max H	Min. Energy	Max. Energy	Static Stroke P_{max}
	[kg]	[kg]	[m]	[m]	[Nm]	[Nm]	[kN]
D 25-32	2,500	700	1.60	3.20	40,000	79,000	14,000
D 30-32	3,000	700	1.60	3.20	48,000	95,000	15,000
D 36-32	3,600	900	1.60	3.20	55,000	114,000	17,000
D 46-32	4,600	900	1.60	3.20	71,000	146,000	20,000
D 62-22	6,200	900	1.70	3.40	107,000	219,000	25,000
D 80-23	8,000	1,100	2.20	3.40	171,000	267,000	31,000
D 100-13	10,000	1,100	2.20	3.40	214,000	334,000	38,000

Table 8.1-8: Technical data of DELMAG diesel hammers with hit dispersion

Type MHF	Weight of Ram	Weight of Pile Cap	min. Energy	Free Drop, max. Energy	Free Drop Stat. Stroke P_{max}	Accelerator, max. Energy	Accelerator Static Stroke
	[kg]	[kg]	[Nm]	[Nm]	[kN]	[Nm]	[kN]
3-4	4,000	900	4,000	40,000	10,000	50,000	11,000
3-5	5,000	900	5,000	50,000	11,000	60,000	12,000
3-6	6,000	900	6,000	60,000	12,000	70,000	13,000
3-7	7,000	1,100	7,000	70,000	13,000	80,000	14,000
5-8	8,000	1,100	8,000	80,000	14,000	95,000	15,500
5-10	10,000	1,200	10,000	100,000	16,000	115,000	17,000
5-12	12,000	1,500	12,000	120,000	18,000	135,000	19,500
10-15	15,000	2,200	15,000	150,000	21,000	175,000	22,500
10-20	20,000	3,000	20,000	200,000	24,000	225,000	25,000

Table 8.1-9: Technical data of MENCK hydraulic drop hammers

For hydraulically operated piling hammers the drop height of the ram can be pre-set as per requirements, ranging usually from about (min.) 15-20 cm to about (max.) 100-120 cm.

It is common practice to provide suitable packing between the hammer's anvil and the pile cap. The choice of packing is depending on the type of pile to be installed, the subsoil conditions and the capacity of piling hammer available for the project. There are some rules to be observed for the right choice of packing. Usually a concrete pile should not be driven with a steel block in the pile cap, as the hammer's stroke will be transmitted directly into the pile head, undamped. This is likely to lead to early destruction of the pile head, even much before reaching the design depth. Steel rope packing (using used steel wire ropes) is easy to arrange at almost any construction site and has proven efficient and effective particularly with difficult, i.e. more dense or hard subsoil conditions. Figure 8.1-1 provides details of typical pile cap packing and some standard factors that can be utilised in computing the suitable piling hammer capacity.

8.1.8.5 Estimation of Pile Penetration per Blow

With the initial pre-selection of a piling hammer type/capacity as per Table 8.1-7, Table 8.1-8 and Table 8.1-9 an approximation can be made regarding the likely rate of pile penetration during the final stage of the pile installation process. Thereby, the following formulae may be applied.

Blow Velocity: $v = \sqrt{2 \cdot g \cdot H}$ [m/s] (Eq. 8.1-5)

Shock Wave Velocity: $a = \sqrt{\frac{E_M \cdot g}{\gamma}}$ [m/s] (Eq. 8.1-6)

284

Impedance of Driving Element: $$Z_p = \frac{E_M \cdot A}{a}$$ [kN·s/m] (Eq. 8.1-7)

Compression of Driving Element During Blow: $$\Delta L = \frac{M \cdot v}{Z_p}$$ [m] (Eq. 8.1-8)

Length of Shock Wave: $$L_w = \frac{\Delta L \cdot E_M \cdot A}{0.5 \cdot P_{max}}$$ [m] (Eq. 8.1-9)

Pile Penetration per Blow: $$s = \frac{\Delta L \cdot (P_{max} - W_e)^3}{2 \cdot P_{max}^2 \cdot W_e}$$ [m] (Eq. 8.1.10)

The symbols are explained in Section 8.1.8.7.

It has to be acknowledged that the piles for standardized permeable groyne structures are horizontally loaded elements. It is important to be aware that any such pile must be driven fully to its designed penetration depth as otherwise the embedded length of the pile will not suffice to carry the occurring horizontal loads during the lifetime of the permeable groyne structure. With a situation at rivers such as the Jamuna River, where severe scouring has to be considered during lifetime of such structures, it is unavoidable to have excessively long piles for installation. With this in mind, it is obvious that the rate of penetration of a pile in its final stage can reach more or less refusal. Therefore, when controlling the results obtained by application of formula (8.1-10), and as a rule of thumb the pile penetration per blow may be considered in a range of about 1 mm.

8.1.8.6 Verification of Selected Piling Hammer with Pile Driving Formula

For the estimation of an appropriate pile driving equipment the pile driving formula by DELMAG is a practical tool, though there are numerous other formulae around the globe. It has to be noted that any such formula can only indicate the order-of-magnitude of such equipment, which final selection much depends also on the experience of the people involved.

Driving capacity of pile: $$W = \frac{E \cdot R}{(s + 0.5 \cdot c) \cdot (R + Q)}$$ [kN] (Eq. 8.1-11)

For explanation of symbols see Section 8.1.8.7.

If no data are available, as a first estimate the elasticity c of the pile and subsoil can be set to c = 0.6 [mm/m] · L for steel and reinforced concrete piles.

The driving capacity of the pile should be at least equal but usually higher than the pile driving resistance to be expected during the pile installation process, i.e. the condition

$$\boxed{W > We}$$

should be fulfilled. There are of course situations where such conditions cannot be met, e.g. in case of non-availability of any suitable piling equipment. In such cases auxiliary measures have to be decided to support the pile installation. Such measures may include emptying soil from the piles' interior in order to reduce the point resistance during driving.

8.1.8.7 Notation Used

Within the forgoing sub-chapters several notations have been used, which are compiled here again in alphabetic order for ease of reference:

A	[cm²]	:	material area of driving element
A_t	[cm²]	:	total area (of piles)
C	[kN]	:	compressive force (concrete piles)
c	[mm]	:	elasticity of pile and subsoil
D	[mm]	:	outer diameter (of piles)
E	[Nm]	:	energy per driving blow
E_M	[kN/m²]	:	modulus of elasticity of driving element
F	[kN]	:	permissible driving force
f_s	[MN/m²]	:	skin friction for penetrometer (CPT)
f_y	[N/mm²]	:	yield stress (steel)
g	[m/s²]	:	gravity acceleration
H	[m]	:	drop height
I_c	[-]	:	consistency index (of cohesive soils)
L	[m]	:	length of pile
l_E	[m]	:	embedment length (of piles)
M	[kN·s²/m]	:	mass of ram
P_{max}	[kN]	:	static stroke of hammer
Q	[kN]	:	weight of pile and pile cap
q_c	[MN/m²]	:	point resistance of penetrometer (CPT)
R	[kN]	:	weight of hammer
s	[mm]	:	penetration of pile per driving blow
T	[kN]	:	tension force (concrete piles)
t	[mm]	:	wall thickness (of steel piles)
t_c	[mm]	:	chosen wall thickness (of steel piles)
U	[cm]	:	circumference (of piles)

2081

W [kN]	:	ultimate load, i.e. driving resistance of pile
W_e [kN]	:	estimated driving resistance (of piles)
γ [kN/m³]	:	specific weight of driving element
σ_p [MN/m²]	:	prestress (in concrete piles)
σ_{sf} [MN/m²]	:	pile tip resistance (of piles)
τ_{mf} [kN/m²]	:	skin friction (of piles)

8.2 PILE INSTALLATION WORKS

8.2.1 General

The pile installation gear shall have an adequately long sturdy leader or proper guides at suitable elevations respectively to permit pitching and installation of the piles in full length. Due consideration must be given in this regard to the type and size of the piles and to any loading on the piles, e.g. due to current flow or driving forces. Only suitable and well fitting driving caps or pile head reinforcements appropriate to the mode of pile installation and to the size and shape of the piles to be installed shall be used. It shall be ensured that even at slight progress of penetration the pile heads are not or only very little deformed.

The installation equipment and any auxiliary means must allow installing the piles to their design depth undamaged.

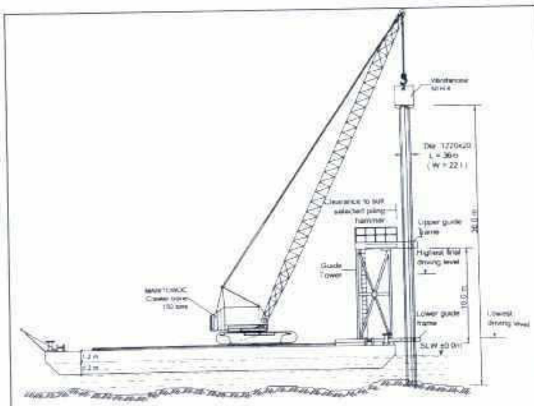
Jetting may be permitted in accordance with applicable rules and standards up to limited depths, if the subsoil permits and other foundations are not endangered. In any case, the last few meters of the piles must be installed without jetting aid.

8.2.2 Tubular Steel Piles

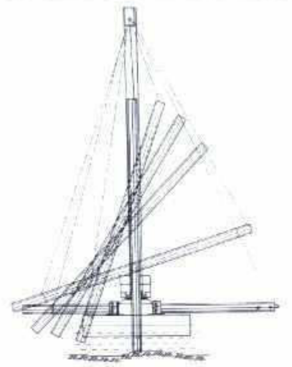
Driving caps shall be of cast steel or heavy welded steel. The packing between the driving cap and the anvil of the driving hammer shall be selected in consideration of the type of the pile to be driven, the encountered soil resistance and the capacity of the driving hammer. In case of timber packing only hardwood shall be used.

For large diameter piles a bell-shaped transition element between the pile head and the driving cap may be used, in order to ensure efficient transmission of the driving energy into the pile to be installed.

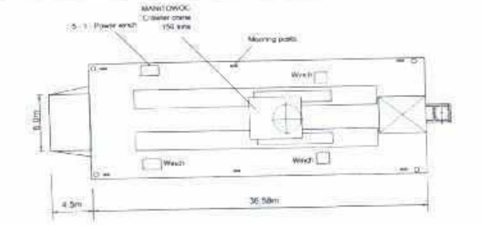
All piles must be installed to their design depth. If, before reaching the design depth, continuing of pile installation is not feasible, other means of installation may be used, such as inside drilling, dredging, air-lifting etc. Reference is made to Section 8.2.7.



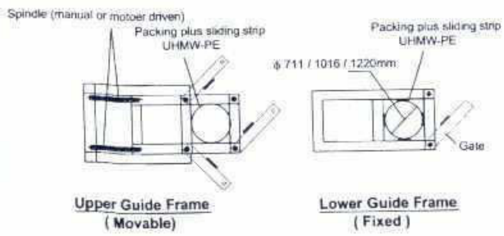
400-t-flat top barge with 150-t-crawler crane and pile guide tower (pile leader)



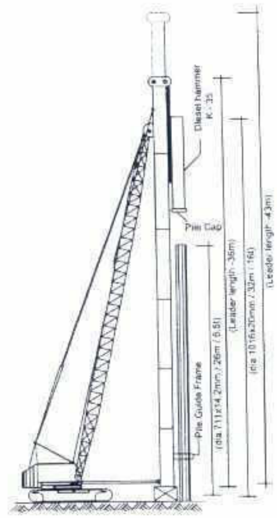
Stages of pile lifting off-shore (analogously to be applied on-shore)



Plan of 400-t-flat top barge for pile installation off-shore



Pile guide frames to be provided at pile installation leader



60-t to 100-t-crawler crane with 100-ft-boom and piling leader l ≥ 36 m (on-shore installation)

Figure 8.2-1: General installation method for large-diameter tubular steel piles

Pile loading shore-to-pontoon

Pile transport to place of installation

Lifting of pile (initial stage)

Final stage of pile lifting

Second stage of pile lifting
(transport pontoon towed away)

Figure 8.2-2: Steel pile installation off-shore

Clamshell dredging for pile installation at the bank-line transition shore-to-riverside

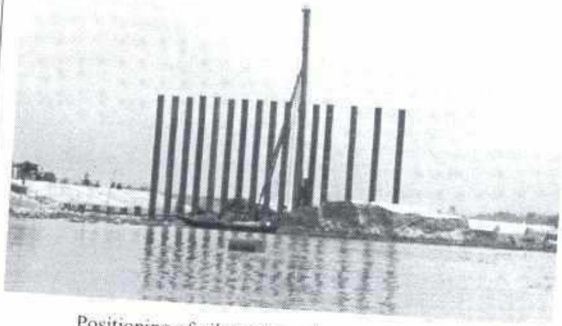
Start of pile installation by vibrator (here: transition zone near bankline)

Pile guide frame

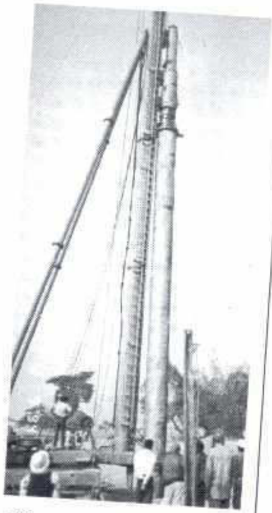
Pile dia. 711 mm during installation stage by vibrator hammer

Final pile installation stage by hydraulic hammer MENCK MHF 10-15

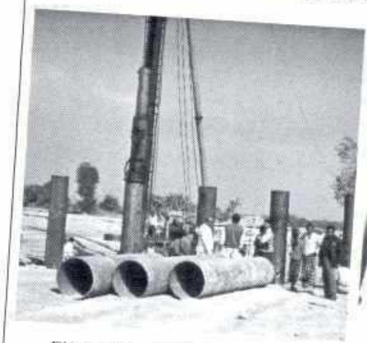
Figure 8.2-3: Steel pile installation works



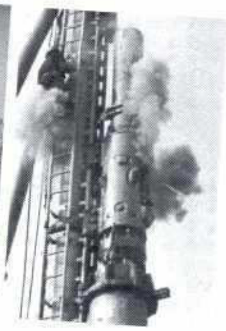
Positioning of pile sections (here: dia. 711 mm), using rail-mounted Kobe-22 pile rig



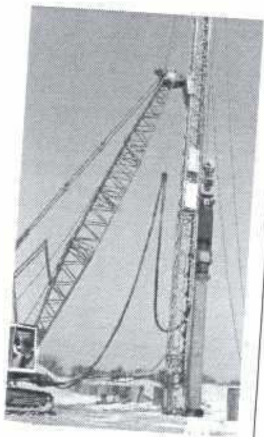
Piling rid Kobe 22 for driving first pile section



Final stage of driving first section



Pile hammer Kobe 22 with pile cap (pile dia. 711 mm)



Final stage of onshore pile installation (employment of hydraulic BSP-hammer with adjustable drop weight 3t to 7t)

Pile extension during pile installation process

Figure 8.2-4: Details of steel pile installation works

278

8.2.3 Steel Sheet Piling

Steel sheet piles must be installed in such a way that a closed wall of maximum safety is achieved. It must be ensured that all sheet-piling will be installed to the required depth as per design without any damages. Special care is to be taken that

- disturbing hindrances at the place of installation are removed
- only straight undistorted sheet piles with sound interlocks are used
- the sheet piles are given satisfactory pitching and guidance during driving
- the driving progress is adapted to the respective local conditions in order to avoid mis-alignment of the sheet piles or jumping out of the interlocks

Driving can be facilitated in suitable soil by jetting. The jetting must be discontinued however, at least 2 m before reaching the design depth. For the remaining distance the pile shall be driven with a hammer in order to re-compact any loosened areas of the soil by vibration, and in order thus to restore the original soil properties.

The driving blow should generally be introduced centrally in the axial direction of the sheet pile element. The effect of the interlock friction, which acts only on one side, may be countered if required by a suitable adjustment of the point of impact.

The sheet pile elements must be guided in such a way, that their design position is achieved in final state. For this, the pile driver itself must be adequately stable, must be firmly emplaced and the leader must always be parallel to the desired inclination of the sheet pile element. The latter shall be guided at least at two points, which are spaced as far apart as possible. A strong lower guide and adequate spacer blocks are especially important. The leading interlock of the pile being driven must also be well guided. When driving without leader, care must be taken to ensure tight contact between the hammer and the sheet pile element by the use of well-fitting leg grips.

The installation of sheet piles shall only be executed in panels, whereby several sheet pile units are pitched and then driven in the sequence e.g. 5-3-1-4-2 and so on.

If a sheet pile or an interlock breaks or a major deviation is determined during its installation, if the designed location is not achieved or driving has not been carried out as per design, additional measures may be required. In case the unit spacing of certain stretches of sheeting must be maintained very accurately e.g. for sheet pile enclosures of cofferdams, the width tolerance must be observed. If necessary, adaptor piles must be inserted. Driving of a taper pile will be required if the sheet piling creeps in the direction of the driving, resulting in an overhang of more than 1:100.

The soundness of the installed sheet piling shall be confirmed by special investigations. Any damage and/or failure in the interlocks and/or other parts of the sheet piling detected in the course of these inspections have to be repaired carefully.

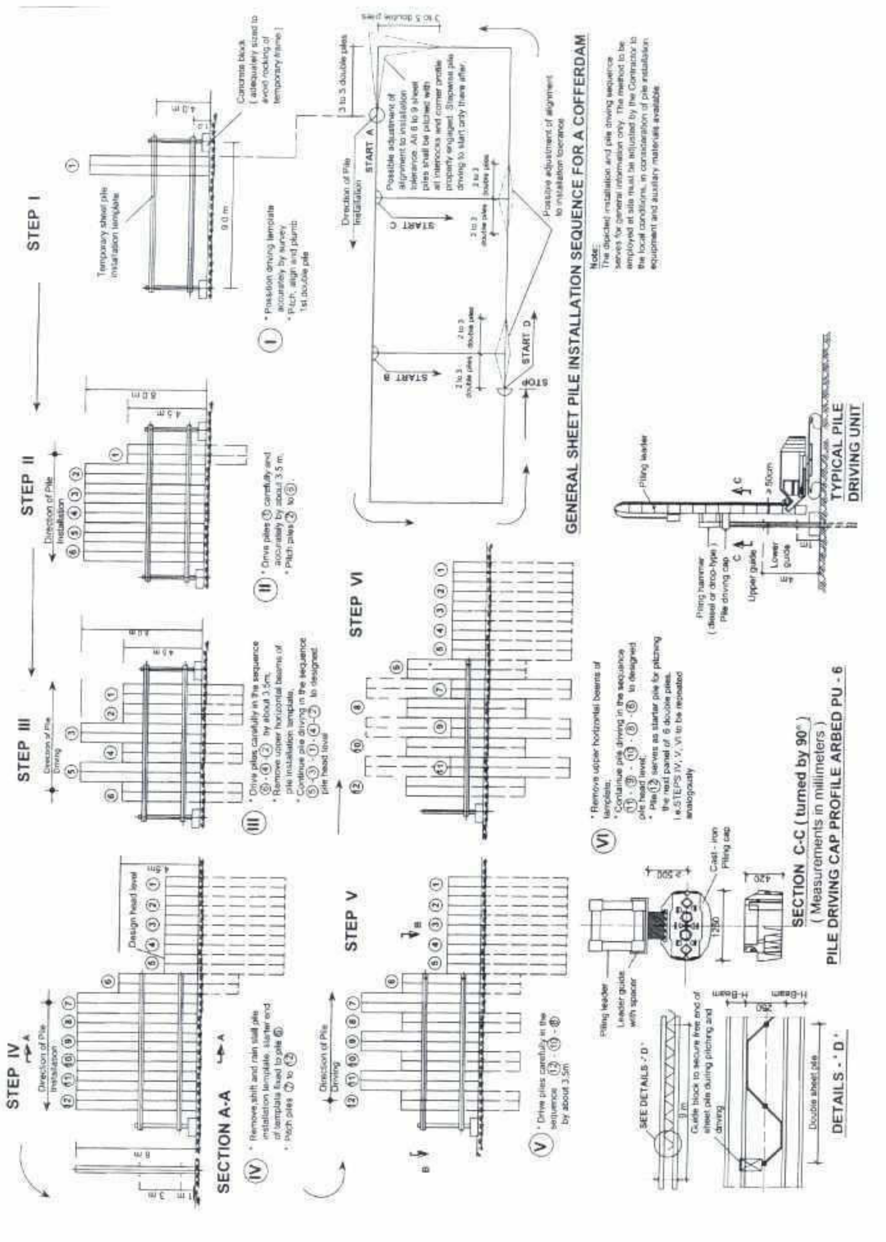


Figure 8.2-5: Principle installation method for sheet-piles

2 a7

8.2.4 Concrete Piles / Concrete Sheet-piles

Concrete piles, either pre-cast reinforced concrete sheet piles or pre-tensioned spun concrete piles, shall be installed only after the concrete has reached the minimum required compression strength of 35 N/mm^2.

Special and tight fitting driving caps shall be used only. The packing must be selected in consideration of the type of pile to be driven. If necessary for safeguarding the pile heads, additional shock-absorbing packings may be provided.

Concrete piles shall be driven to the design depth. If, before reaching the design depth, a continuation of pile installation is not feasible, other installation means may be used. Reference is made in this regard to Section 8.2.7.

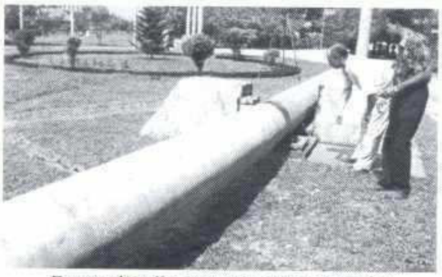
8.2.5 Pile Butts

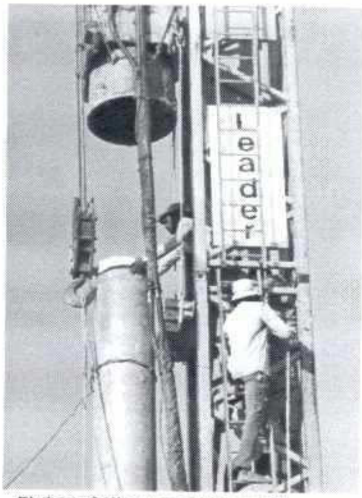
8.2.5.1 Steel Piles and Steel Sheet Piling

This Section applies to the providing of pile joints at the pile welding yard or during pile installation as well as for reinforcement of pile points or pile heads. Only steel qualities equivalent to the respective structural element/steel pile shall be used. Welders qualified for the work and who possess a welding certificate as per relevant standards shall only carry out all welding work. Trial welds and tests shall confirm the soundness of the welding procedure.

Welded joints shall withstand driving stresses as well as dynamic loads during the lifetime of the structures. Therefore, the welds and in particular the root welding must be carried out with special care, free of defects. Where the top of a driven steel pile must be fitted with a welded joint, it shall not be placed in areas with driving deformations. In such cases, the top end must be cut off to a point well below the limit of deformation, or to at least 10 cm below the pile top in case of no visible deformations. Meeting ends of pile sections to be joined shall be true and formed with a clean cut perpendicular to the pile axis. The meeting ends shall be made with clean cuts to form a V-profile in cross section through the metal.

Butt welds shall be checked by ultrasonic inspection. Fillet welds shall be inspected by magnetic particle inspection. An independent inspection company shall carry out all testing of welds. The standards of acceptability by which welding will be judged shall be in accordance with relevant international standards. Welds that are considered defective, are to be cut out, to be remade and re-tested.


Factory bending test of welded pile joint



Fitting of pile cap packing and hammer helmet (hydraulic BSP-hammer 357-7, 7 t drop weight)


Prestressed spun concrete piles joined at site (2 sections @ 10 m, total length 20 m)


Final driving stage with RB-driving rig and hydraulic BSP-hammer 357-7


Initial driving stage with RB-driving rig and hydraulic BSP-hammer 357-7

Figure 8.2-6: Spun concrete pile installation on-shore (here: diameter 500 mm, length 20 m)

2 (F) ✓

8.2.5.2 Concrete Piles

Pile butts are required for long piles depending on the transport facilities. They shall be of standard design and in accordance with internationally recognized standards.

For jointing the pile ends, special temporary pile clamps must be employed to ensure perfect axial jointing of the pile. The joints are made of steel. Only certified welders shall carry out all welding of pile joints. Welding thickness shall be 12 mm. The steel elements of the completed and tested pile joints are to be provided with a first-class corrosion protection, as far as they are remaining above SLW in the final pile position.

Testing of completed welds shall be done by color-penetration or equivalent method. Any defects or imperfections in the weld shall be repaired. In case of any doubt, the respective area shall be expertly grinded-out and re-welded and tested. An independent inspection company, or an equivalent institution shall carry out all testing of welds.

8.2.6 Surveying and Tolerances

The piles shall be set up at locations as shown on the drawings and pitched with the greatest accuracy. The location of the piles shall be determined with the help of high precision surveying instruments, by using permanent and verified base points/bench marks.

During pile installation, the alignment of the piles shall be controlled by appropriate measures throughout that process. In case of any increasing deviation from the designed location, the installation process shall be interrupted and the pile realigned. Immediately after installation of a pile and removal of any guides, the final location of the pile shall be surveyed with reference to fix points and the adherence to tolerances shall be checked.

After installation of piles, the location tolerances must be limited to the most possible minimum value, in order to correspond to the allowable tolerance prescribed by the type of the respective structural element and the designed alignment of the piles. The position of the pile heads shall not exceed the following tolerances

(a) in the horizontal plane:
- along groyne axis: ± 10 cm
- perpendicular to groyne axis: ± 20 cm

(b) elevation of pile head: ± 5 cm

8.2.7 Observations During Installation

8.2.7.1 Obstacles

The term "obstacle" refers to unforeseen artificial obstructions which cannot be reasonably traced by surface inspections and subsoil obstructions e.g. rock layers, boulders in the ground, cemented soil layers and the like, as far as these cannot be derived from the available data or by any samples or information which could also have been exhibited from available borings or trial holes, and which cannot be penetrated with the appropriate equipment as required.

Prior to the installation of piles, the surface in the area where the structures are to be set up must be searched through divers for obstacles, which would prevent sound installation of piles.

8.2.7.2 Observations of Pile Behaviour

If during installation of a pile the pile behaviour becomes questionable e.g. sudden reduction or increase in the penetration depth per measuring interval, moving of the pile head or the like, the installation procedure must be interrupted immediately. The reasons for this behaviour are to be investigated and suitable countermeasures be implemented.

8.2.7.3 Non-Reaching of Design Depth

If the progress of pile driving reduces considerably but uniformly before reaching the design depth, and if this is not due to exceptional circumstances e.g. encountering of an obstacle, the driving of the pile has still to be continued as long as the penetration is

- more than 10 cm with 80 blows (steel piles), and
- more than 2 cm with 10 blows (concrete piles)

at maximum permissible driving energy for the pile. If a procedure other than driving is used for the installation of the piles, corresponding criteria are to be determined.

8.2.7.4 Damages to Piles

Should a pile suffer any breakage, other heavy damages or unacceptable misalignments, it has to be extracted and to be replaced. Thereby it must be proven that the subsoil in the respective area is compactable and suitable measures are to be taken, in order to re-establish the load-bearing capacity of the soil being disturbed by pile extracting. Extraction of any ill-driven pile can only be permitted if other piles are not influenced in their bearing capacities.

8.2.8 Records of Installation

All piles shall be given a penetration marking for recording of the driving logs and supervision of the driving operation. The intervals shall be every half and full meter, referred to the pile point. In consideration of the increasing driving resistance to be expected with different soil strata and at the design depth, marking intervals shall be decreased to 0.1 m in the upper pile length. Records of the progress of the pile installation work shall be carried out which, among others, must contain the following data:

General

- date, starting and completion time of driving
- name of observer
- pile No., type and weight
- designed pile length and level of pile head
- location of pile and level of terrain at pitching point

Pile Installation Process

- employed equipment, type, weight and capacity of driving hammer
- applied drop height/energy of the hammer at individual stages of installation process
- number of blows per decimeter of pile penetration during the entire driving operation, respectively vibration period per decimeter
- total number of blows

Results

- achieved driving depth
- final level of pile and pile point
- pile inclination and deviation of pile head position after installation

Special Incidents

- cutting length, if any
- extension length of pile, if butted under the driving hammer
- position of such butt referred to pile point
- effective pile length
- comments on special incidents, interruptions etc. (e.g. encountered obstacles, unexpected soil layers etc.)

Before starting installation, a standardized log sheet has to be prepared, which takes regard of the specific requirements of the site.

For driving of sheet piles, similar records have to be done.

8.3 MANUFACTURE AND SUPPLY OF PILES

8.3.1 Steel Piles

8.3.2.1 General

For the manufacture of steel piles, the supplier has to prepare the complete shop drawings and to submit the same to the Engineer.

8.3.2.2 Steel Qualities

Tubular foundation steel piles shall be of specially skilled steel of grade St 37-3, or equivalent highly weldable steel quality of the required brittle-fracture strength:

- Tensile strength β = 340-470 N/mm^2
- Yield point:
  min β_s = 235 N/mm^2 ($t \leq 16$ mm)
  min β_s = 225 N/mm^2 (16 mm $< t \leq 40$ mm)

The chemical and mechanical properties of the steel pile material shall conform to DIN 17120.

8.3.2.3 Manufacturing and Tolerances

Diameter and unit weight of welded steel piles shall conform to DIN 2458.

Manufacturing and tolerances shall conform to DIN 17120, insofar as no other directives are prescribed in this Section.

Length tolerance of the specified steel piles shall be - 0 / +100 mm.

8.3.2.4 Welding

The pile manufacturer has to prepare and submit to the Engineer a complete Welding Procedure Specification for execution of site welds, including recommendation of a range of suitable welding materials.

The manufacturer of welded tubular steel piles shall possess a proof of competence to weld structural steel as per DIN 18800, Part 7, or an equivalent standard or regulation.

Only welders tested and certified as per DIN 8560 or equivalent standard, for the type of welding to be executed shall be employed for this work. Relevant certificates are to be handed over to the Engineer.

Welding shall be in accordance with DIN 17120. The welding process shall be confirmed by welding procedure tests, whereby the recommendations of the steel plate manufacturer shall be given due consideration.

Spiral welding seams shall only be formed by automatic submerged arc welding, whereby at least one pass shall be made on the inside of the pipe.

All welding seams shall be welded 100%. The welding seam quality factor shall be 1.0.

All welding seams shall be tested as described in Section 8.3.2.5. Imperfections or defects in welding seams shall be repaired as far as technically feasible, subject to non-destructive testing.

8.3.2.5 Tests and Acceptances

Tubular steel pile sections shall be inspected and tested as per DIN 17120 and delivered with a works test certificate as per DIN 50049-3.1B.

Welding super elevation shall be within the limits of DIN 8563, Part 3, evaluation group AS.

Non-destructive inspection by ultrasonic method shall cover the full length of all welds. The standard of acceptability for evaluation of non-destructive inspection shall follow DIN 8563, sheet 3, evaluation group AS.

The chemical composition of the steel plate material shall be confirmed by ladle analyses. It must meet the requirements set against the steel qualities as per Section 8.3.2.2.

Failed tests shall be repeated as per DIN 17120. If the conditions stipulated therein cannot be fulfilled, the entire lot of steel pile sections will be rejected.

The Employer must have the right to have the material and welding tests supervised by an independent inspector at the pile manufacturer's plant.

All tubular steel pile sections manufactured in conformity with these Specifications shall be provided with identification markings by the manufacturer as follows:

- Manufacturer's name or mark
- Size in mm
- Steel grade

Any additional marking as deemed useful by the manufacturer or Employer may be agreed upon later on and shall be provided by the manufacturer without additional costs to the Employer.

No material or tubular steel pile section shall leave the manufacturer's plant unless concurrence of the test and inspection results with the Specifications has been confirmed through presentation of the works test certificates to the Employer.

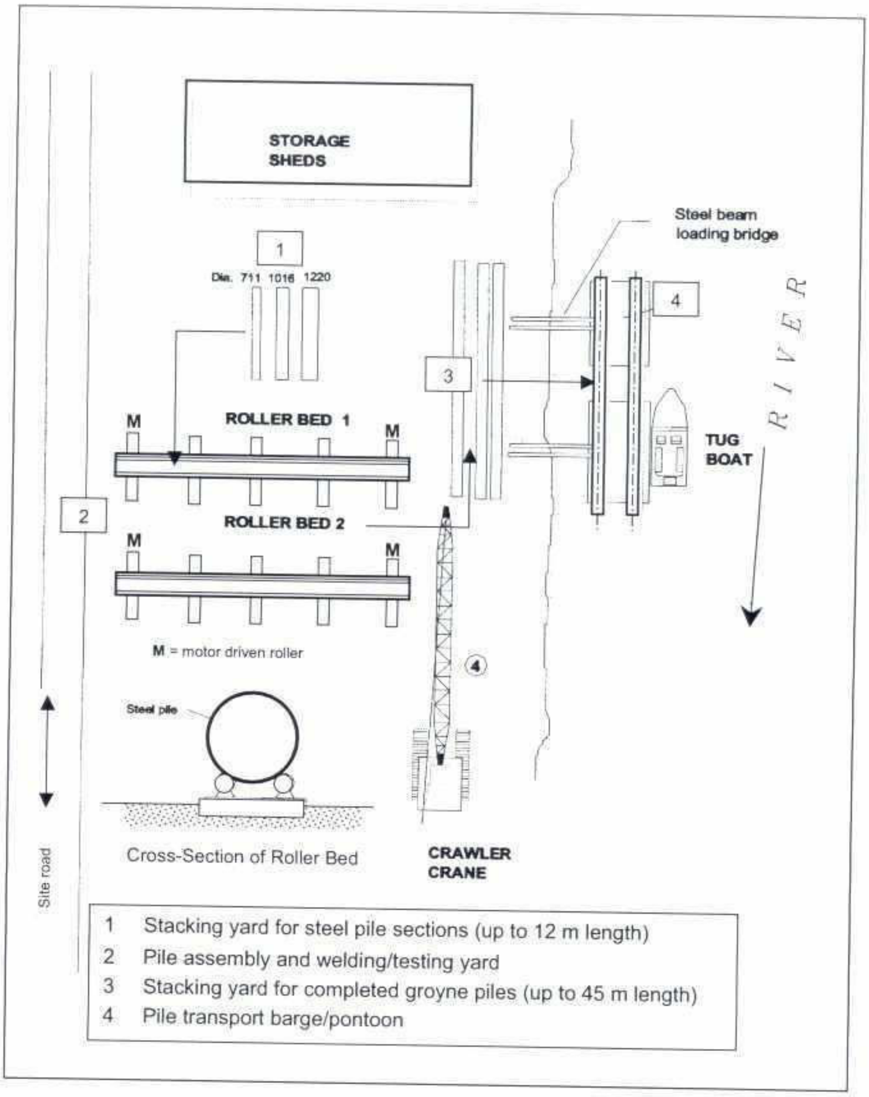
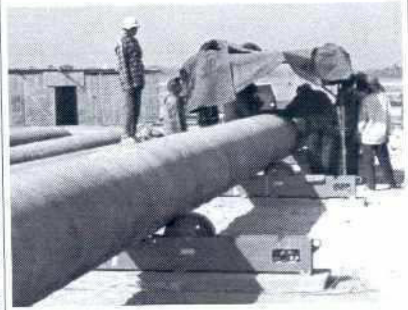


Figure 8.3-1: Typical layout of a pile assembly and finishing yard

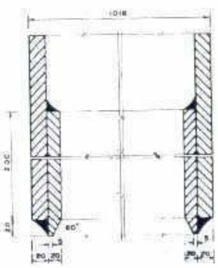
Roller bed for pile section alignment



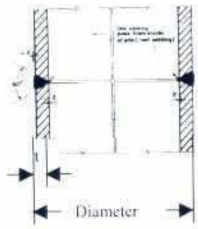
Welding under shelter

Adjustment of pile sections for joining

Joint grinding prior to next weld pass



Typical Pile Point Reinforcement



Typical Pile Butt Joint at Site Welding Yard

Figure 8.3-2: Typical details of welding works

8.3.2.6 <u>Delivery</u>

The tubular steel pile sections shall be delivered in the lengths and with prepared ends, as indicated in the pile delivery schedule to be prepared by the Employer. Special pile point reinforcements shall be completed at the pile manufacturer's plant in accordance with Section 8.3.2.8.

8.3.2.7 Preservation Coatings

The tubular steel pile material for standardized groyne structures do not normally need to receive a corrosion protection coating as their steel wall thickness covers sufficiently for corrosion loss.

If corrosion protection is to be provided, may be for esthetic reasons or as warning mark, it is recommended to use one-component polyurethane coatings, as these can be applied under any condition of high humidity.

For any coating to be applied, the steel surfaces must be de-rusted by shot-blasting, to comply with Grade Sa 2.5.

8.3.2.8 Pile Point Reinforcements

Reinforcing plating shall be of steel grade St 37-3, DIN 17100.

Welding material shall be coordinated with the manufacturer of the steel plate material for the steel piles. All welding work shall only be carried out by welders qualified for the work and who possess a welding certificate as per DIN 8560.

Trial welds and tests to the satisfaction of the Employer shall confirm the soundness of the welding procedure selected by the manufacturer.

Fillet welds shall be inspected by magnetic particle inspection. The standards of acceptability by which welding will be judged shall be in accordance with DIN 8563, sheet 3, evaluation group BK (fillet welds).

Welds that are considered defective are to be cut out and the welds have to be remade and retested to the satisfaction of the Employer.

8.3.2 Prestressed Spun Concrete Piles

8.3.2.1 General

Prestressed spun concrete piles can only be produced by specialized and well experienced factories. Production facilities and continued production control are to be such as to guarantee the desired quality.

Prestressed spun concrete piles shall be manufactured in accordance with DIN 4026 and DIN 4227 or JIS A-5335/JIS B-3536, or equivalent international standard.

8.3.2.2 Materials

Cement shall conform to ASTM C-150, Type I. A works test certificate must accompany each cement consignment used for the pile production. In the absence of certificates, samples of cement shall be taken from the consignment. These shall be forwarded for analysis and testing to a laboratory acceptable to the Employer.

Aggregates shall conform to the requirements of DIN 4226 and DIN 1045 or ASTM C 33. Combined aggregates shall be within the limits of grading curves "A" and "B", as specified in DIN 1045.

Concrete Quality shall conform to DIN 1045, Class B 45. It shall be of low shrinkage, and the w/c-ratio be between 0.45 and 0.48.

The minimum cement content shall be 375 kg per 1 m^3 ready-mixed concrete. The concrete must be extremely dense and watertight.

The concrete mix design must be confirmed by suitability tests according to DIN 1045 and DIN 1048. The relevant test reports shall be submitted to the Employer prior to pile production.

If there is a change in the source of supply of cement or aggregates or if the quality of cement or aggregates should change considerably, suitability tests for a new concrete design mix must be repeated and results submitted to the Employer.

During production of prestressed spun concrete piles, quality tests as per DIN 1045 shall be carried out. Six test cubes of 20 x 20 x 20 cm shall be prepared for each pile to check the compressive strength. Compaction of concrete test specimen shall correspond to the employed pile production method. Alternatively cylindrical test specimen may be used.

Three each of the test specimen shall be tested after 7 days and 28 days of its production. All test results must at least correspond to the following values (reference to 20 x 20 x 20 cm test specimen):

- Minimum compressive strength of each cube after 28 days: 45 N/mm^2.
- Series strength (minimum average compressive strength of each series of cubes) after 28 days: 50 N/mm^2.

The steel moulds for the spun process must be suitable for the specified pile dimensions and designed to ensure perfectly straight piles, true to diameter and shape.

Reinforcement steels must comply with ASTM A-615 or equivalent standard.
The high tensile steel wires shall be of 7 mm diameter and the grade must guarantee a tensile strength of 1600 N/mm^2.

Material certificates shall be submitted to the Employer, along with factory acceptance certificates as per DIN 50049-3.1B or equivalent standard.

Prior to incorporation in the works, the high tensile steel wires must be checked by the pile manufacturer with regard to strength reducing defects. Wires damaged in transit (e.g. notches, etc.) or showing corrosion (except airborne rust) shall under no circumstances be used in the works.

8.3.2.3 Production

Pile diameter shall be 500 mm with a wall thickness of 100 mm. Length of each pile unit 10 m, both ends prepared for butt jointing.

Formworks shall be set absolutely straight. Piles that do not fulfill these requirements or with damaged pile ends will not be accepted.

The concrete cover of the tensioning wires shall be 50 mm.

The concrete shall be compacted by spinning process. The manufacturer shall prove to the Employer that the selected method produces a concrete quality of the desired strength and watertightness.

The Contractor shall provide suitable protection of the freshly poured concrete and use appropriate measures such as steam curing.

All piles shall be permanently marked with a production number and the date of concreting for identification.

8.3.2.4 Prestressing

The high tensile steel wires shall be pre-tensioned prior to concreting the piles.

The tensioning system and equipment shall be particularly suitable for pre-stressed concrete piles. Relevant documents in proof of their suitability and reliability shall be submitted to the Employer.

The hydraulic stressing jack and pump shall be equipped with calibrated high precision pressure gauges to measure the applicable tensile force.

The stressing equipment shall be verified to the Employer by a certificate verifying the internal friction losses at various pressure stages, as well as with a calibration certificate for the pressure gauge.

The calibration of the stressing equipment shall be carried out whenever required to the Employer's satisfaction.

The initial stress of the individual wires prior to occurrence of the slip at anchoring shall not exceed 80 % of the ultimate tensile strength of the wires. The slip of wires at anchoring shall not exceed 7 mm.

The pre-tensioning loads shall be transferred to the piles, only when the concrete has reached a minimum strength of 32 N/mm². The fulfillment of this requirement must be confirmed by compressive strength tests.

8.3.2.5 Transport and Storage

The manufacturer shall paint marks for lifting, transportation and storage of the piles, all to the Employer's satisfaction.

Pre-stressed concrete piles shall not be handled and transported unless the concrete has reached a compressive strength of at least 35 N/mm², which must be verified by cube crushing tests.

8.3.2.6 Pile Butts

Pile butts must guarantee a statically perfect compression, tension and bending proof connection and meet all requirements of the pile driving operation. Pile butts shall be of standard design and in accordance with internationally recognized standards, such as JIS A-5335/JIS B-3536. Relevant design documents are to be submitted to the Employer.

8.3.2.7 Tests and Acceptances

The pile manufacturer shall provide for and execute all factory tests as required under this Section, in consideration of the applicable Standards.

The pile manufacturer shall prepare and submit to the Employer detailed reports on pile production, containing all relevant data concerning concreting, pre-stressing, curing, including all results of quality control. The type of report shall be coordinated with the Employer.

At least two piles of 10 m length each and one pile joint (two pile sections of 10 m each, butt jointed) shall be subject to bending test as per DIN 4227 at the manufacturers plant. The tests shall be carried out in the presence of the Employer.

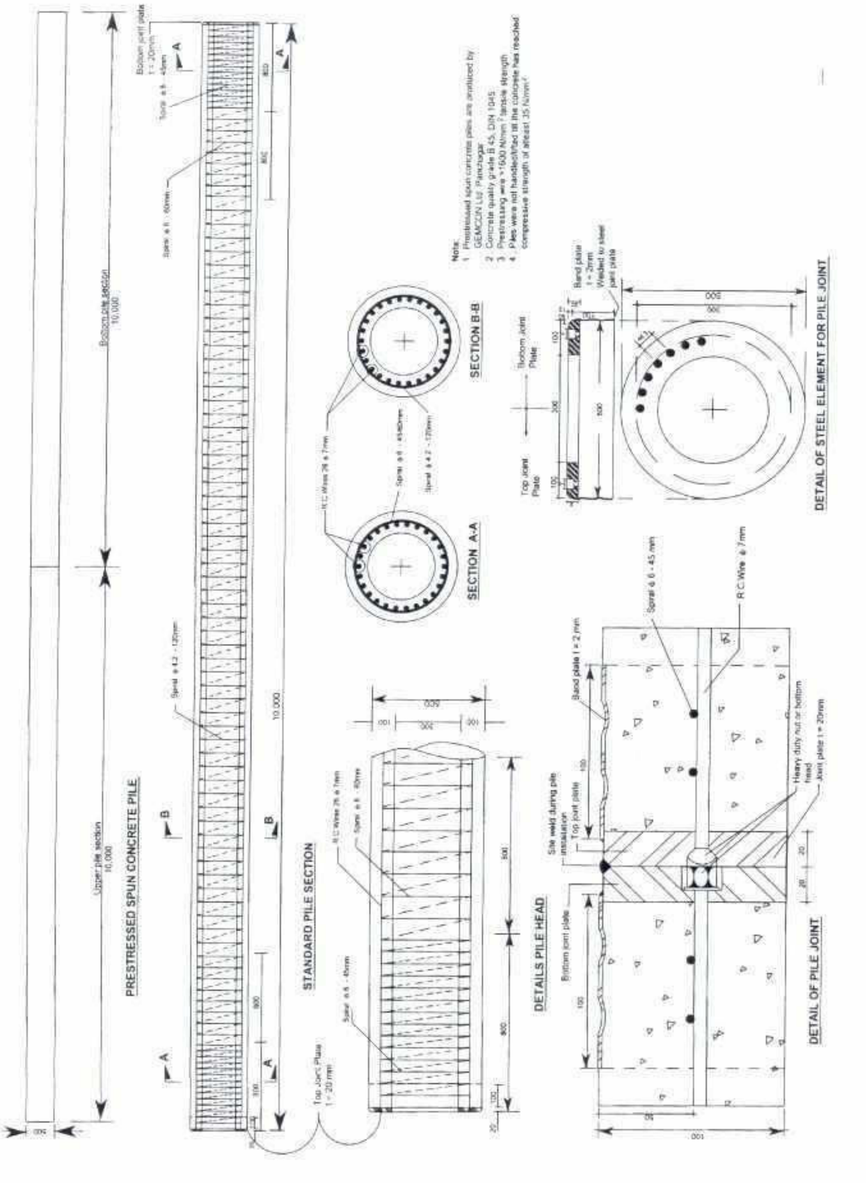


Figure 8.3-3: Typical design for a pre-stressed spun concrete pile section

290

8.3.3 Reinforced Concrete Sheet Piles

8.3.3.1 General

Reinforced concrete sheet piles can only be produced by well experienced factories. Production facilities and continued production control are to be such as to guarantee the desired quality. Reinforced concrete sheet piles shall be manufactured in accordance with DIN 1045 or equivalent standard.

8.3.3.2 Materials

Cement shall conform to ASTM C-150, Type I. A works test certificate must accompany cement consignments used for the sheet pile production. In the absence of certificates, samples of cement shall be taken from the consignment. These shall be forwarded for analysis and testing to a laboratory acceptable to the Employer.

Aggregates shall conform to the requirements of DIN 4226 and DIN 1045 or ASTM C 33. Combined aggregates shall be within the limits of grading curves "A" and "B", as specified in DIN 1045.

Concrete Quality shall conform to DIN 1045, Class B 45. It shall be of low shrinkage, and the w/c-ratio be between 0.45 and 0.48. The minimum cement content shall be 375 kg per 1 m^3 ready-mixed concrete. The concrete must be extremely dense and watertight.

The concrete mix design must be confirmed by suitability tests according to DIN 1045 and DIN 1048. The relevant test reports shall be submitted to the Employer prior to pile production.

If there is a change in the source of supply of cement or aggregates or if the quality of cement or aggregates should change considerably, suitability tests for a new concrete design mix must be repeated and results submitted to the Employer.

During production of reinforced concrete sheet piles, quality tests as per DIN 1045 shall be performed. Six test cubes of 20 x 20 x 20 cm shall be prepared for each pile to check the compressive strength. Compaction of concrete test specimen shall correspond to the employed pile production method. Alternatively cylindrical test specimen may be used.

Three each of the test specimen shall be tested for compression strength after 7 days and 28 days of its production. All test results must at least correspond to the following values (reference to 20 x 20 x 20 cm test specimen):

- Minimum compressive strength of each cube after 28 days: 45 N/mm^2
- Series strength (minimum average compressive strength of each series of cubes) after 28 days: 50 N/mm^2.

Formwork for the piles shall be prepared and cast in a strong and robust steel formwork. The manufacturer must submit detailed drawings of the intended formwork, to the Employer's satisfaction. The number of formwork sets must correspond to the production output required as per agreed delivery schedule.

Reinforcing steel shall be deformed bars of Grade 60, ASTM A-615 or equivalent standard. Material certificates shall be submitted to the Employer.

8.3.3.3 Production

Size of sheet piles may be about 490 x 250 mm, length as specified in the Drawings (common range 8 m to 12 m each), with special pile point and grove and tongue joints.

Formworks shall be set absolutely straight. The pile tip and grove and tongue joints must be perfectly shaped. Piles that do not fulfill these requirements or with damaged pile point or grove/tongue joints will not be accepted.

The concrete cover of the main reinforcing bars shall be 30 mm (50 mm for marine environment).

The concrete shall be preferably compacted by external formwork vibrators. The manufacturer shall submit to the Employer relevant details regarding make, capacity, number employed and fixing arrangement of the vibrators.

The manufacturer shall provide suitable protection of the freshly poured concrete and use appropriate measures such as steam curing.

All piles shall be permanently marked with a production number and the date of concreting for identification.

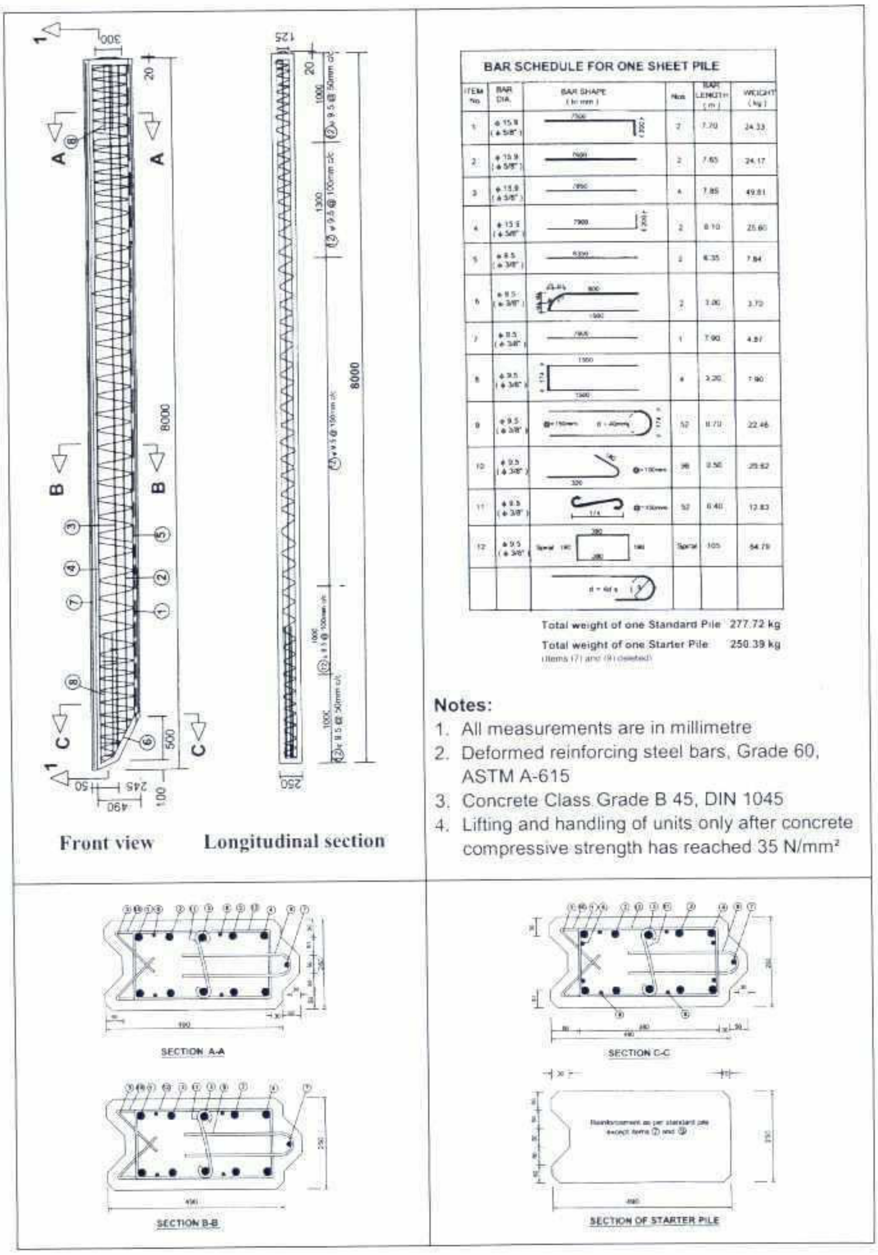
8.3.3.4 Transport and Storage

The manufacturer shall paint marks for lifting, transportation and storage of the piles, all to the Employer's satisfaction.

Reinforced concrete sheet piles shall not be handled and transported unless the concrete has reached a compressive strength of at least 35 N/mm^2, which must be verified by cube crushing tests.

8.3.3.5 Tests and Acceptances

The pile manufacturer shall provide for and execute all factory tests as required under this Section, in consideration of the applicable Standards. The pile manufacturer shall prepare and submit to the Employer detailed reports on pile production, containing all relevant data concerning concreting, curing, including all results of quality control. The type of report shall be coordinated with the Employer.



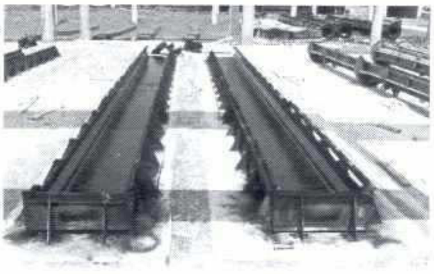
BAR SCHEDULE FOR ONE SHEET PILE

ITEM No.	BAR DIA.	BAR SHAPE (in mm)	Nos.	BAR LENGTH (m)	WEIGHT (kg)
1	φ 15.9 (φ 5/8")	7500, 200	2	7.70	24.33
2	φ 15.9 (φ 5/8")	7600	2	7.65	24.17
3	φ 15.9 (φ 5/8")	7850	4	7.85	49.81
4	φ 15.9 (φ 5/8")	7900, 200	2	8.10	25.60
5	φ 9.5 (φ 3/8")	6300	2	6.35	7.84
6	φ 9.5 (φ 3/8")	800, 1500	2	3.00	3.70
7	φ 9.5 (φ 3/8")	7850	1	7.90	4.87
8	φ 9.5 (φ 3/8")	1500, 174, 1500	4	3.20	7.90
9	φ 9.5 (φ 3/8")	174	52	0.70	22.46
10	φ 9.5 (φ 3/8")	320	98	0.50	29.62
11	φ 9.5 (φ 3/8")	174	52	0.40	12.83
12	φ 9.5 (φ 3/8")	Spiral 190, 380	Spiral	105	54.79

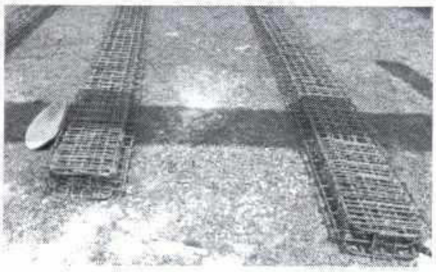
Total weight of one Standard Pile 277.72 kg
Total weight of one Starter Pile 250.39 kg
(items (7) and (9) deleted)

Figure 8.3-4: Typical design of reinforced concrete sheet-pile unit (here: 490 x 250 x 8000 mm)

296



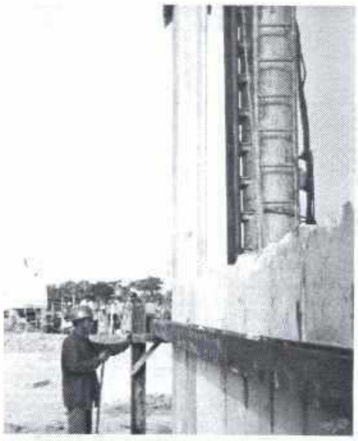
Steel moulds for concrete sheet piles at factory



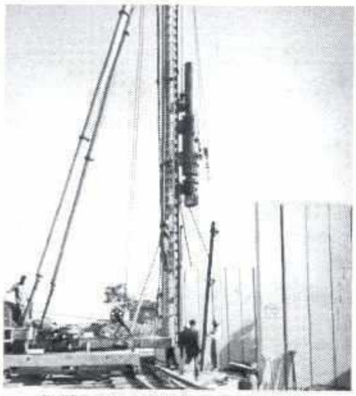
Reinforcement cages for concrete sheet piles

Unloading of RCC-sheet piles for transhipment

Cofferdam: installation of anchors and capping beam



Pitching and alignment of concrete sheet piles for driving



Rail-mounted piling rig Kobe 22

Cofferdam: backfilling and compaction

Figure 8.3-5: Details of concrete sheet pile installation

8.3.4 Bored Concrete Piles (In-situ Concrete Piles)

8.3.4.1 Definition

Bored piles within the meaning of this Section are in-situ concrete piles as per DIN 4014 or equivalent standard, which are constructed by sinking a steel casing with a nominal diameter equal to the designed diameter of the pile. The soil inside the pipe is removed by means of appropriate boring tools, while the pipe is simultaneously rotated or driven and advanced into the ground.

After the steel casing is sunk to the desired depth the borehole must be cleaned, a reinforcing cage be installed and the empty space be concreted. Thereby the bore pipe is reclaimed to a specified depth as a rule, but shall serve as permanent protective casing in the specified upper free length of the pile.

The Employer may approve alternative boring methods, but a permanent protective casing is to be provided in the specified upper pile length in any case.

In the following, the term "diameter" will always be construed as the outer diameter of the bore casing.

8.3.4.2 General

Only such firms/sub-contractors shall be entrusted with the construction of bored piles who possess a thorough knowledge and extensive experience in this special field.

With the Tender, the Tenderer shall submit full details on experience and equipment of the firm, who shall execute the bored pile works.

The responsible construction superintendent must be thoroughly acquainted with the type of bored pile construction and its execution.

Only experienced and reliable foremen, who have already successfully executed such work, shall supervise bored pile works.

Piece-work shall not be tolerated by the Employer because of the therewith connected sources of danger for the quality of the bore pile construction. The same holds good for night work, which will be approved by the Employer only in exceptional cases.

8.3.4.3 Location and Tolerances

The bore pipes must be surveyed-in and positioned with utmost accuracy. The Contractor shall provide only such equipment inclusive of suitable auxiliary means and devices that enable exact positioning besides technically sound execution of the boring work.

The tolerance in the location of pile heads shall not exceed 100 mm in any direction. The deviation of the finally prepared pile heads shall not deviate more than 50 mm from the design level.

The inclination of the pile centerline shall not deviate from the given direction by more than 1%.

If the tolerances are exceeded due to Contractor's fault, the Contractor must bear all the resulting costs and supplies required for the technically sound rectification of such discrepancies, such as construction of additional piles, etc., all to the satisfaction of the Employer.

8.3.4.4 Materials

Reinforcement shall be of deformed bars of Grade 60 as per ASTM A-615 or equivalent Standard.

The reinforcement shall be fabricated as a reinforcement cage in accordance with the Drawings. The cage is to be so inserted in the bore pipe, that it is not displaced during concreting, and that it cannot be lifted when the bore pipe is extracted.

The reinforcement cage must be firmly placed in position in the bore pipe through spacers. The concrete cover of the outer reinforcement shall be at least 60 mm.

The basic materials for producing the concrete shall conform to the respective Standard, e.g. DIN 1045.

Cement shall be Ordinary Portland Cement, Type I, as per ASTM C-150 or BS 12 or equivalent Standard. Rapid-setting cement shall not be used.

The in-situ concrete for the bored piles shall correspond to Grade II, Class B 35, DIN 1045. It must contain at least 400 kg cement per 1 m^3 ready-mixed concrete, with a grain size distribution of combined aggregates between 0 - 16 mm.

The cement content shall be increased to 450 kg per 1 m^3 ready-mixed concrete for the first pour to be filled into the tremie pipe.

The design mix for concrete Class B 35 shall be verified by preliminary tests as stipulated by DIN 1045. The mix must be a most possible dense concrete with a finest grain proportion (cement plus aggregate of size up to 0.2 mm) of more than 400 kg per 1 m^3 ready-mixed concrete.

The fresh concrete shall have a consistency with flowing properties, but the spreading index (consistency range as per DIN 1045) shall be limited to 55 to 60 cm. The water-cement ratio shall not exceed the value of 0.60.

28

Quality control shall be ensured by routine tests, analogously to DIN 1045. For each day of concreting the bored piles 6 test cubes of size 20 x 20 x 20 cm shall be prepared, of which three each shall be tested after 7 and 28 days.

The concrete strength after 28 days shall correspond at least to

- Minimum compressive strength of each cube: 35 N/mm²,
- Minimum average compressive strength of each series of 3 cubes: 40 N/mm².
-

8.3.4.5 Boring Work

The drilling equipment shall be of mechanical or hydraulic drive of adequate capacity in consideration of the soil to be penetrated and the boring depths. It must be particularly suitable for the drilling method to be employed.

Details of Contractor's drilling method, intended number, type and capacity of gear and drilling tools shall already be submitted to the Employer along with the Tender and be supplemented as required in adequate time before commencing the boring works.

The Employer will not tolerate a drilling method and/or equipment by which no technically sound execution of the boring work will be ensured.

Bored piles shall be sunk to design depth as indicated in the Drawings and Bill of Quantities or as otherwise directed by the Employer.

If soil layers are encountered in which technically sound construction of the concrete piles and their load bearing capacity is doubtful, the Contractor must immediately inform the Employer hereof. Any further work or measures shall be carried out only in coordination with the Employer.

With bored piles, which are staggered according to the depth, the deeper seated piles must be constructed first. The Employer will permit no exception to this general principle, since the soil under already constructed piles would be subsequently weakened in its bearing capacity.

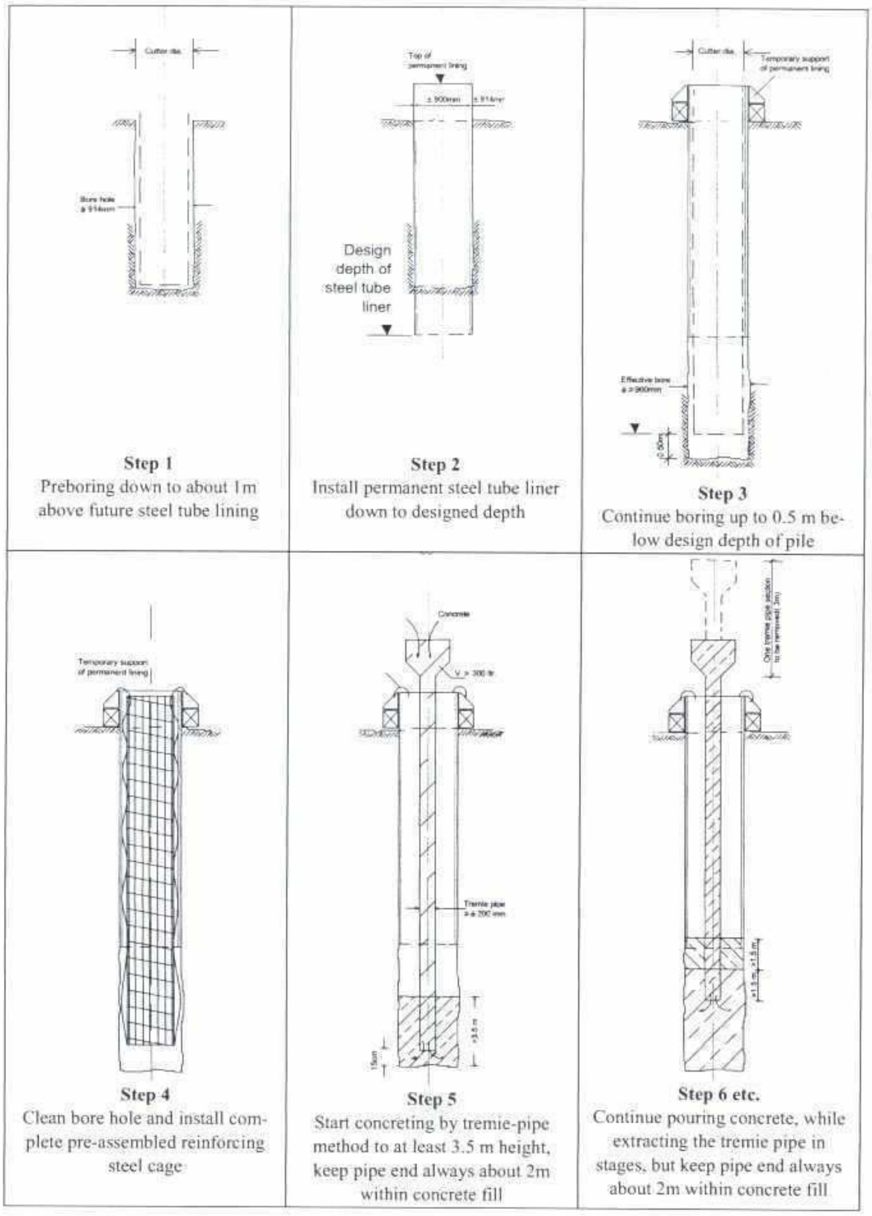
The outer diameter of the bore pipes must correspond to the nominal diameter of the in-situ bore piles.

The pipe joints shall be flush inside and outside. Joints shall be screwed and must be watertight. If necessary, suitable sealing compound shall be used for the joints.

The outer diameter of the cutting edge of the bore pipe may be larger than that of the bore pipe, however, up to maximum 20 mm only.

Bore pipes which are designated to become an integral part of the in-situ concrete piles (i.e. as permanent protective casing) shall be of the size and quality as indicated in the Bill of Quantities and may be provided with welded joints.

299



Step 1
Preboring down to about 1m above future steel tube lining

Step 2
Install permanent steel tube liner down to designed depth

Step 3
Continue boring up to 0.5 m below design depth of pile

Step 4
Clean bore hole and install complete pre-assembled reinforcing steel cage

Step 5
Start concreting by tremie-pipe method to at least 3.5 m height, keep pipe end always about 2m within concrete fill

Step 6 etc.
Continue pouring concrete, while extracting the tremie pipe in stages, but keep pipe end always about 2m within concrete fill

Figure 8.3-6: General installation method for bored in-situ cast reinforced concrete piles

These bore pipes must be adequately supported and secured during the entire bore pile construction, until the concrete has achieved adequate strength.

The Contractor is obligated to submit the detailed proposal for these temporary works to the Employer

For drilling and discharging only such equipment and tools shall be used which are particularly suitable for cutting the individual soil layers and which extensively avoid loosening of the surrounding soil.

Jetting aid will not be permitted for sinking the bore pipes. Any drilling equipment or tools causing far-reaching and thus adverse loosening outside the bore pipe will not be allowed for the construction work by the Employer, e.g. above all such tools, whose effectiveness depends exclusively on the principle of suction.

The bore pipe must advance ahead of the boring tool/core discharge as far as at all possible but at least 300 mm to 500 mm in non-cohesive, fine grained soils, whether drilling above or below ground water table.

Deviation from this condition will only be tolerated in firm, cohesive soils, cemented layers etc., which do not or permit only to a minor extent the advancing of the bore pipe.

If the foregoing provision is not being sufficiently adhered to at any time by the Contractor, those bore holes will not be accepted by the Employer for construction of an in-situ concrete pile and must be abandoned at the Contractor's expense.

When drilling below water table, works shall be carried out under continuous water supply to the bore pipe to ensure maintaining of an adequately high water table in the bore hole at any stage of boring, i.e. during drilling and on withdrawal of the drilling tool.

The excess pressure inside the bore pipe shall be at least 1.0 m. If confined ground water is to be expected, the bore pipe must be filled so high with water before reaching the water-bearing layer, that the pressure of the confined water is kept at least at equilibrium, in order to surely prevent a buoyancy or flushing out of soil particles.

The hydrostatic pressure difference in the bore pipe should be determined in consideration of the highest possible artesian pressure head. In cases of doubt, drilling must be carried out from the very beginning with an adequately high water column.

The aforementioned is of particular importance if sands which tend to flow are encountered during drilling, or such silty or cohesive soils which quickly change their consistency during drilling under water influx.

Thereby shortly occurring under pressure against the ground water can already lead to damaging soil uplifts in the bore pipe. When passing through the aforementioned soil strata, care must also be taken, that upward movements of the drilling tool in the bore pipe are carried out relatively slowly, in order to avoid additional suction.

Concreting of a bored pile has to be executed immediately on the day of completing the drilling. If this condition cannot be met for any reason, the boring works must be suspended at least 2 m above the designed pile tip level. The final boring to design depth shall be completed only on the day of concreting, immediately prior to the concreting.

Any borehole that must be abandoned is to be filled with non-cohesive sand, which shall be slightly compacted in order to restore at least the original condition of the subsoil.

8.3.4.6 Obstacles during Drilling

The term "obstacles" within this Section shall mean unforeseen artificial obstructions which cannot be readily traced by inspection or taken from relevant documents (drawings e.g. old underground structures, service lines etc.), and subsoil obstructions e.g. rock layers, boulders in the ground, cemented soil layers and the like which can be neither penetrated with the appropriate equipment as required under the Contract nor derived from the data provided in the Contract or by any samples or information which could also have been exhibited by the Contractor from available borings or trial holes.

The Employer is to be notified immediately, when obstacles are encountered during drilling. All subsequent measures shall be coordinated with the Employer.

Smaller obstacles may be removed by appropriate means and tools, but loosening of the soil must be avoided to the possible extent. Jetting aid or blasting inside the bore hole is strictly forbidden.

If before reaching the design depth obstacles are encountered which cannot be removed without extensive soil loosening, the borehole must be abandoned in any case. The borehole is then to be filled with lean concrete or with non-cohesive sand, so that at least the original condition of the soil is restored to the Employer's satisfaction.

The Employer will decide on the location of substitute bore holes.

8.3.4.7 Concreting

The sequence of concreting the bored piles must be coordinated so that the setting of the concrete is not impaired through the work on neighboring piles.

Immediately after completion of the drilling, the reinforcement cage must be inserted and the pile then concreted.

It is not permitted to drill a number of bores to design depth and to start the inserting of the reinforcement cage and the concreting thereafter

Special, extraordinary circumstances may prevent the maintaining of the above provision. The Employer is to be informed of such situation immediately and the subsequent construction measures shall be coordinated with him. In any case, it must be determined prior to concreting of such boreholes whether the bottom level of the bore point has lifted in the meantime. Under certain circumstances, re boring must be carried out before the concreting. For checking a possibly lifted contact surface, the previously inserted reinforcement cage must be withdrawn.

The reinforcement cage must be raised by 50 mm to 100 mm above the bottom at the beginning of the concreting, so that its bottom end is also sufficiently encased with concrete.

The concrete must be poured by tremie pipe method, so that it reaches the bottom of the pile point, thus ensuring that it becomes neither segregated, interrupted, contracted nor impured, and that it is given a dense texture.

The tremie pipes must be joined watertight. The pipe diameter shall be selected in consideration of the concrete consistency and borehole depth. The tremie pipes must extend down to the pile point.

During progress of the concreting operation, it must be ensured that the concrete column does not break during withdrawal of the tremie pipe and that no water can enter the tremie pipe. The stepwise withdrawal of a tremie pipe section shall not be done unless the tremie pipe reaches down by at least 3.0 m into the already placed concrete, and at any stage the minimum embedment of the tremie pipe into the concrete column shall be 2 m.

Enough fresh concrete must always be held available and placed, so that the bored piles can be filled by at least 2.0 m in height in one operation.

The concrete may be compacted according to the progress of placing, if required by the bore pile construction method employed by the Contractor, in order to fulfill the Specifications.

The bore pipes shall be withdrawn slowly and uniformly, particularly in the upper length of the pile, to avoid definitely that the concrete column breaks or is contracted. During extracting the concrete column shall always be kept at least 1.5 m above the bore pipe end, to maintain an adequate pressure against incoming ground water or soil.

Bore pipes which are an integral element of the in-situ concrete piles shall be withdrawn only to the elevation shown on the Drawings or as otherwise directed by the Employer, and their top end shall be cut at design elevation.

Flushing out of binding agents of the fresh concrete through flowing ground water must be avoided with certainty. If this provision cannot be followed for special reasons, the Contractor must inform the Employer thereof before the start of concreting. As the case may be, the bore pipes shall then remain in the ground, or the concreting be postponed.

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8.3.4.8 Records of Installation

The Contractor is obligated to prepare detailed records on the construction of every bored pile and to submit them to the Employer in the required number of copies.

The form of reports shall meet the requirements of DIN 4014 and must be coordinated with the Employer. A specimen form is attached at the end of this Section.

8.3.4.9 Controls and Acceptances

The Contractor must verify the concrete consumption for each in-situ concrete pile. Immediately after completion of the concreting the actually measured and placed quantity is to be compared with the theoretical cubic content of bore hole and must be at least equal. The actual data shall be entered in the form for the construction of bored piles and be submitted to the Employer.

Bored piles, for which the Contractor cannot provide the foregoing verification will not be considered as permanent structural members and be rejected by the Employer. In such cases, the Contractor shall install a replacement pile as directed by the Employer and the Contractor is not entitled to demand compensation for his performances.

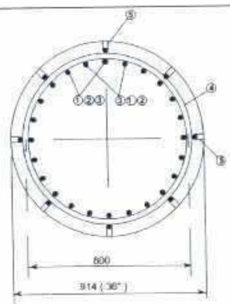
Concrete test cubes shall be tested at due dates. The results shall conform at least to the criteria stipulated in DIN 1045 and shall be entered in the final bored pile construction reports to be submitted to the Employer.

Independently of the quantity of concrete placed within a day, six test cubes shall be prepared for each pile produced. Three cubes each shall be tested for compressive strength after 7 and 28 days, to the requirements of Table 8.3-1.

Class	Characteristic Strength (minimum compressive strength f each test cube) β_{W28}	Series strength (minimum average compressive strength of each series of three cubes) β_{Wm}
B 25	25 N / mm^2	30 N / mm^2
B 35	35 N / mm^2	40 N / mm^2

Table 8.3-1: Required strength of concrete for bored piles

282



Pile cross-section

REINFORCEMENT BAR SCHEDULE (Steel Grade 60)

ITEM No.	BAR DIA (mm)	BAR SHAPE (in meter)	Nos	CUTTING LENGTH (m)	WEIGHT (kg)
1	25 (1")		24	5.50	506.20
2	25 (1")		24	10.50	970.20
3	25 (1")		24	11.00	1016.40
4	ϕ 9.5 (ϕ 3/8")	0.20 0.80 Spiral		310.00	172.05
5	□ 50 x 4 mm	0.80 0.063	64	0.80	80.38
Total Weight Per Pile					2747.23

Reinforcement bar schedule
(for one pile dia. 36", length 22 m)

General Notes:

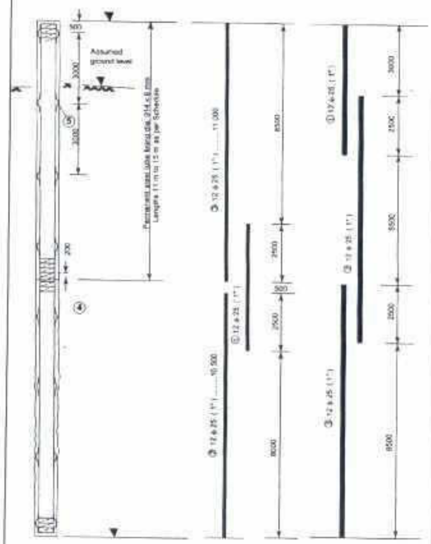
1. Measurements are shown in millimetres, except shown otherwise.
2. Reinforcing steel bars shall be Grade 60 as per ASTM A-615.
3. Coarse concrete aggregates shall be only natural gravel or crushed stone, maximum aggregate size 32 mm.
4. Concrete shall meet requirements of Class B 35 (DIN 1045). Minimum cement content 400 kg/m³ of ready mixed concrete.
5. Water-cement ratio maximum 0.6.
6. Permanent steel tube lining shall be provided within the anticipated total exposure length of the piles (after erosion has taken place). Steel grade St 37 or equivalent.

Figure 8.3-7: Typical cross-section of bored in-situ cast reinforced concrete pile (here D=914 mm)

Drilling for in-situ cast reinforced concrete piles by reverse rotary method

Concreting of pile by tremie pipe method

Lifting of reinforcement cage for installation in the completed bore hole



Typical reinforcement design of cast in-situ bired pile dia. 36" (914 mm)

Figure 8.3-8: Bored in-situ cast reinforced concrete pile works on-shore

(PROJECT NAME)

BORED PILE PRODUCTION REPORT

Pile Location No.:

Bored Pile Production No.:

Date:

1. BASIC DATA

Contractor: ______

Construction Site: ______

Reference Drawings: ______

1. Pile Data

1.1 Nominal Diameter of Pile: ______ mm

1.2 Casing Data:

- temporary casing: ______ mm
- permanent casing: ______ mm

1.3 Drilling Tool (type): ______

1.4 Outer Diameter of

Drilling Tool : ______ mm

Cutting Edge : ______ mm

2. Reinforcement of Pile

2.1 Reinforcement Drawing No.: ______

2.2 Installation of Reinforcement Cage:

before concreting ☐

after concreting ☐

2.3 Spacers

Type: ______

Spacing in Longitudinal Direction: ______ mm

3. Concrete Mix

3.1 Type (Strength Class): B ______

3.2 Consistency/Slump: ______ cm

3.3 Mixing Plant

Type: ______

Capacity: ______ m^3/mix

3.4 Cement

Manufacturer: ______

Type: ______

3.5 Quantity per 1 m^3 Concrete: ______ kg/m^3

3.6 Aggregates:

Combined Aggregate Curve No.:

______

Max. Grain Size: ______ mm

3.7 Water-Cement Ratio: ______

3.8 Concrete Additives:

Type/Brand Name: ______

% of Cement Content: ______ (by Volume)

3.9 Retarder:

Type/Brand Name: ______

Retard Time: ______ h

4. Concreting Procedure

4.1 Means of Placing Concrete

Tremie Pipe ϕ : ______ mm

Pumping Pipe ϕ : ______ mm

Other means

Description: ______

4.2 Measures to Clean Bore Hole Bottom

______

______

4.3 Measures to Separate Concrete from Bentonite

Suspension at the Start of Concreting:

______

______

5. Remarks/Comments

______

______

☐ Tick-mark where applicable

Figure 8.3-9: Report form for bored in-situ cast reinforced concrete pile works (part I)

2+8

Error! Bookmark not defined. **(PROJECT NAME)**

BORED PILE PRODUCTION REPORT

Page 2 of 3

II. VARIABLE BORED PILE PRODUCTION DATA

Date of Boring : ____________

Date of Concreting : ____________

Pile Location No. : ____________

Bored Pile Production No. : ____________

Soil Profile				
Depth		Type of Soil	Ground Water Table	Length of Casing
Below Ground Level [m]	Referred to ± 0.0 m		Below Ground [m]	from...to...m

1. Pile Data

1.1 Check of Borehole Depth:

__________ m below ground level

1.2 Bit drilling from __________ m

to ________ m below ground level

1.3 Deviations from Designed Location:

X-Axis ________ cm

Y-Axis ________ cm

2. Data of Bentonite - Suspension (actual measured data)

	At Start of Concreting	After Concreting
τ_F		
ρ_F		

Level of Bentonite - Suspension (actual measured data):

_____ m above bottom level of casing

_____ m above ground water table

3. Reinforcement of Pile

3.1 Deviations from Reinforcement Drawing No.: ____________

3.2 Alteration of Pile Length: ______ m

3.3 Alteration of Reinforcement (Reason): ____________

3.4 Other Alterations: ____________

4. In-Situ Concrete

Special Observations: ____________

5. Concreting

5.1 Level of Suspension at Start of Concreting: ______ m above lower edge of casing

5.2 Verification of Concrete Consumption:

Theoretical: ____________ m^3

Actual consumed: ____________ m^3

Fig. 8.3-9: Report form for bored in-situ cast reinforced concrete pile works (part II)

(PROJECT NAME) BORED PILE PRODUCTION REPORT	Page 3 of 3 Pile Location No.: Bored Pile Production No.:

6. Bored Pile Construction Time

Action	Surrounding Temperature °C	Execution Date		Execution Time	
		Start	End	Start	End
Drilling					
Installation of Permanent Casing					
Bit drilling					
Interruption					
Concreting					

7. Concrete Compressive Strength Tests

7.1 Size of Test Specimen: ____________________

7.2 Identification Numbers of Test Specimen: ____________________

7.3 Compressive Strength Test Results

ID - No.	7-days test [N/mm²]
Average :	

ID - No.	28-days test [N/mm²]
Average :	

8. Remarks

Deviations from Basic Data : ____________________

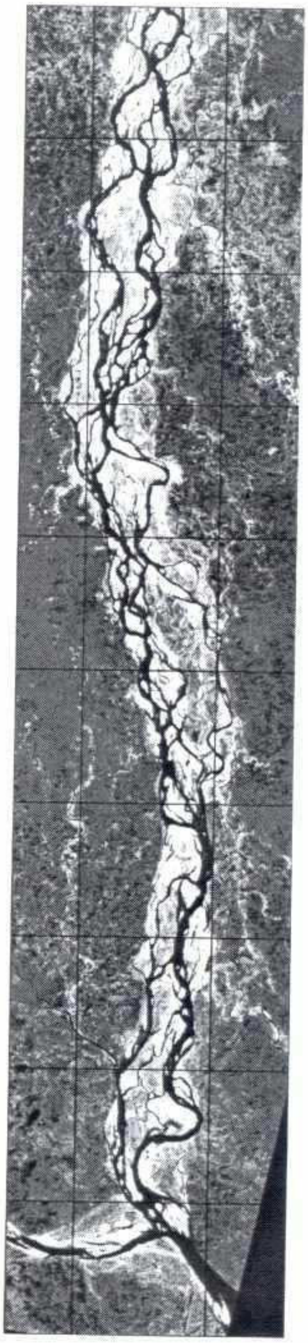
9. Signatures

Contractor's Drilling Master: ____________________ Date: ___/___/___

Contractor's Superintendent: ____________________ Date: ___/___/___

Engineer's Representative: ____________________ Date: ___/___/___

Fig. 8.3-9: Report form for bored in-situ cast reinforced concrete pile works (part III)



PART 3

DESIGN PLATES

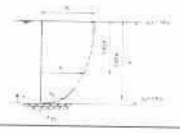
PART 3 - DESIGN PLATES

Contents

2b2

Cube-shaped concrete blocks
(hand-placed in single layer)

Design against current



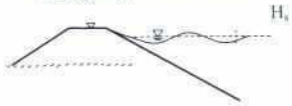
Minimum Size $D_n \geq \frac{\bar{u}^2}{2g} \cdot \frac{0.035}{\psi_{cr}} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{\Delta_m \cdot K_S}$

D_n	[m]	size (arris length) or thickness of single layer cc-blocks
$\bar{u}$	[m/s]	depth averaged flow velocity if replaced by $u_b = 0.6\,\bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
g	[m/s²]	acceleration due to gravity (= 9.81)
ϕ_{SC}	[-]	stability factor for current
ψ_{cr}	[-]	critical shear stress parameter
K_t	[-]	turbulence factor
K_h	[-]	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2 \cdot \left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	[-]	bank normal slope factor $K_S = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for B.D. rivers and a conservative assumption [1)]
α	[°]	slope angle of bank or structure
ε_s	[°]	angle of repose considering the material specific internal friction

Coefficient	Unit	Value	Coefficient	Unit	Value
Turbulence intensity K_t			**Stability factor ϕ_{SC}**		
Normal turbulence in rivers	[-]	1.0	Continuous Protection	[-]	0.65
Non-uniform flow with increased turbulence, mild outer bends	[-]	1.5	Exposed edges, transitions	[-]	1.25
High turbulence, local disturbances, sharp outer bends	[-]	2.0			
Angle of repose ε_s			**Critical shear stress parameter ψ_{cr} (Shields)**	[-]	0.05
Geo-textile	[°]	20			
Granular	[°]	25			
Material density ρ					
Concrete (khoa aggregates) ρ_c	[kg/m³]	2000			
Water ρ_w	[kg/m³]	1000			

Cube-shaped concrete blocks
(hand-placed in single layer)

Design against waves



Minimum Size $D_n \geq \dfrac{H_S \cdot \xi_z^b}{\Delta_m \cdot \psi_u \cdot \phi_{SW} \cdot \cos\alpha}$ $(\xi_z < 3, \cot\alpha \geq 2)$

D_n	[m]	size (arris length) or thickness of single layer cc-blocks
H_s	[m]	significant wave height
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
ϕ_{SW}	[-]	stability factor for wave loads
ψ_u	[-]	system specific stability upgrading factor
ξ_z	[-]	wave similarity parameter $\tan\alpha \cdot \dfrac{1.25 \cdot T_m}{\sqrt{H_S}}$
α	[°]	slope angle of bank or structure
T_m	[s]	mean wave period
b	[-]	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

Coefficient	Unit	Value
Stability factor for incipient motion ϕ_{SW}	[-]	2.25
Stability upgrading factor ψ_u	[-]	1.50
Interaction Coefficient b	[-]	0.67

Cube-shaped concrete blocks
(hand-placed in single layer)

Supplementary Information/ Specifications/ Construction Details

Supplementary Information

[1] slope factor K_s

Embankment						
	slope	1 : 2.5	1 : 3	1 : 3.5	1 : 4	1 : 5
	angle	21.8°	18.4°	15.9°	14°	11.3°
CC-blocks on geo-textile filter mat (ε_s = 20°)		—	0.385	0.599	0.707	0.820
CC-blocks on granular filter (ε_s = 25°)		0.477	0.665	0.761	0.820	0.886

Specifications and Construction Details

To increase the stability of such a blockwork, the gaps between individual cc-blocks should filled with smaller gravel material (increased interlocking effect).

The material density of concrete units made from coarse khoa aggregates may vary between ρ_c = 1 ,980 to 2,000 kg/m^3 dependent on the quality of the aggregates and the mixture of concrete. Adequate care and control during concrete production is very important for the stability of the individual element. Frequent analysis of the physical stability of test specimen is required (see figure below).

Figure: Testing of concrete specimen

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Rip - Rap Revetments

(Randomly placed broken stones or boulders)

Design against current

$$\text{Diameter } D_n \geq \frac{\bar{u}^2}{2g} \cdot \frac{0.035}{\psi_{cr}} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{(1-n) \cdot \Delta_m \cdot K_S}$$

D_n	[m]	characteristic diameter of boulders or broken stone material [5)]
$\bar{U}$	[m/s]	depth averaged flow velocity if replaced by $u_b = 0.6\,\bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
G	[m/s²]	acceleration due to gravity (= 9.81)
ϕ_{SC}	[-]	stability factor for current
ψ_{cr}	[-]	critical shear stress parameter
K_t	[-]	turbulence factor
K_h	[-]	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2 \cdot \left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	[-]	bank normal slope factor $K_S = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for B.D. rivers and a conservative assumption [1)]
α	[°]	slope angle of bank or structure
ε_s	[°]	angle of repose considering the material specific internal friction

Coefficient	Unit	Value	Coefficient	Unit	Value
Turbulence intensity K_t			**Stability factor ϕ_{SC}**		
Normal turbulence in rivers	[-]	1.0	Continuous Protection	[-]	0.75
Non-uniform flow with increased turbulence, mild outer bends	[-]	1.5	Exposed edges, transitions	[-]	1.25 (1.50)
High turbulence, local disturbances, sharp outer bends	[-]	2.0			
Angle of repose ε_s			**Critical shear stress parameter ψ_{cr} (Shields)**	[-]	0.035
Geo-textile	[°]	20			
Granular	[°]	25			
Material density ρ					
Boulders ρ_c [2)]	[kg/m³]	2600			
Water ρ_w	[kg/m³]	1000			

226

Rip - Rap Revetments

(Randomly placed broken stones or boulders)

Design against waves

Diameter $D_n \geq \frac{H_S \cdot \xi_z^b}{\Delta_m \cdot \psi_u \cdot \phi_{SW} \cdot \cos\alpha}$ ($\xi_z < 3$, $\cot\alpha \geq 2$)

D_n	[m]	characteristic diameter of boulders or broken stone material [5]
H_s	[m]	significant wave height
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w)/\rho_w$
ϕ_{SW}	[-]	stability factor for wave loads
ψ_u	[-]	system specific stability upgrading factor
ξ_z	[-]	wave similarity parameter $\tan\alpha \cdot \frac{1.25 \cdot T_m}{\sqrt{H_S}}$
α	[°]	slope angle of bank or structure
T_m	[s]	mean wave period
b	[-]	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

Coefficient	Unit	Value
Stability factor for incipient motion ϕ_{SW} [3]	[-]	2.25
Stability upgrading factor ψ_u [4]	[-]	1.0
Interaction Coefficient b	[-]	0.5

[3] for maximum tolerable damage of a two-layer system on granular filter $\phi_{SW} = 3.0$

[4] for a two layer rip - rap system (no damages); in case certain damages are tolerated the upgrading factor might be increased to $\psi_u = 1.33$

Rip - Rap Revetments

(Randomly placed broken stones or boulders)

Supplementary Information/ Specifications/ Construction Details

Supplementary Information

[1] slope factor K_s

Embankment	Slope	1 : 2.5	1 : 3	1 : 3.5	1 : 4	1 : 5
	Angle	21.8°	18.4°	15.9°	14°	11.3°
Rip-rap on geo-textile filter mat ($\varepsilon_s = 20°$)		—	0.385	0.599	0.707	0.820
Rip-rap on granular filter ($\varepsilon_s = 25°$)		0.477	0.665	0.761	0.820	0.886

[2] In average; material density may vary between 2,400 and 2,800 kg/m^3, dependent on the source.

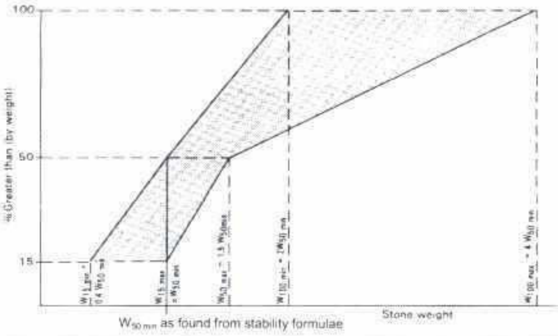
Specifications and Construction Details

[5] For broken stone material: nominal diameter $D_{50} = D_n / 0.85$. For stones/ boulders this relation may be used tentatively only and should be verified for detailed design.

In case of surface or pattern grouting (refer to Chapter 7) the nominal diameter found from wave loads (which are in general decisive for the stability of grouted material) can be reduced to

$D_{50\,(grout)} = 0.9\, D_{50\,(rip\text{-}rap)}$ for surface grouting
$D_{50\,(grout)} = 0.6\, D_{50\,(rip\text{-}rap)}$ for pattern grouting

The typical grading envelop for rip – rap material (recommended by PIANC, 1987) is shown in the following diagram (refer also to Chapter 7). W_{50}: weight of a single unit of nominal diameter.

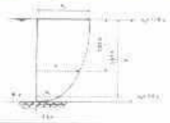


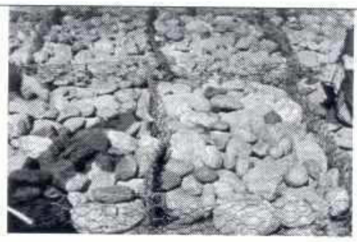
$$W_{50_{min}} = D_n^3 \cdot \rho_s \qquad [kg]$$

$$= \left(\frac{D_{50}}{0.85}\right)^3 \cdot \rho_s \quad [kg]$$

Stone filled mattress systems

Design against current





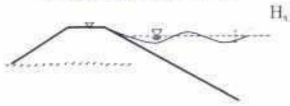
Minimum thickness of mattress $D_n \geq \frac{0.035 \cdot \bar{u}^2}{(1-n)\cdot\Delta_m \cdot 2g} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{K_S \cdot \psi_{cr}}$

D_n	[m]	thickness of mattress system
$\bar{u}$	[m/s]	depth averaged flow velocity if replaced by $u_b = 0.6\,\bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
n	[-]	volume of voids in the mattress fill (approx. n = 0.4)
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w)/\rho_w$
g	[m/s²]	acceleration due to gravity (= 9.81)
ϕ_{SC}	[-]	stability factor for current
ψ_{cr}	[-]	critical shear stress parameter
K_t	[-]	Turbulence factor
K_h	[-]	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2\cdot\left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	[-]	bank normal slope factor $K_S = \sqrt{1-\left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for B.D. rivers and a conservative assumption [1]
α	[°]	slope angle of bank or structure
ε_s	[°]	angle of repose considering the material specific internal friction

Coefficient	Unit	Value	Coefficient	Unit	Value
Turbulence intensity K_t			**Stability factor ϕ_{SC}**		
Normal turbulence in rivers	[-]	1.0	Continuous Protection	[-]	0.50
Non-uniform flow with increased turbulence, mild outer bends	[-]	1.0	Exposed edges, transitions	[-]	1.00
High turbulence, local disturbances, sharp outer bends	[-]	1.0			
Angle of repose ε_s [2]			**Critical shear stress parameter ψ_{cr} (Shields)**	[-]	0.07
Geo-textile	[°]	20			
Granular	[°]	25			
Material density ρ_s					
Stone fill	[kg/m³]	2600			

Stone filled mattress systems

Design against waves



Minimum thickness of mattress $D_n \geq \dfrac{H_S \cdot \xi_z^b}{(1-n)\cdot \Delta_m \cdot \psi_u \cdot \phi_{SW} \cdot \cos\alpha}$; ($\xi_z < 3$, $\cot\alpha \geq 2$)

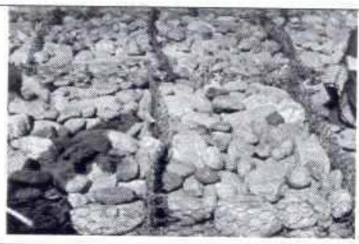
D_n	[m]	thickness of mattress system
H_s	[m]	significant wave height
n	[-]	volume of voids in the mattress fill (approx. n = 0.4)
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
ϕ_{SW}	[-]	stability factor for wave loads
ψ_u	[-]	system specific stability upgrading factor
ξ_z	[-]	wave similarity parameter $\tan\alpha \cdot \dfrac{1.25 \cdot T_m}{\sqrt{H_S}}$
α	[°]	slope angle of bank or structure
T_m	[s]	mean wave period
b	[-]	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

Coefficient	Unit	Value
Stability factor for incipient motion ϕ_{SW}	[-]	2.25
Stability upgrading factor ψ_u	[-]	2.50
Interaction Coefficient b	[-]	0.50

209

Stone filled mattress systems

Supplementary Information/
Specifications/ Construction Details



Supplementary Information

1) slope factor K_s

Embankment	slope	1 : 2.5	1 : 3	1 : 3.5	1 : 4	1 : 5
	angle	21.8°	18.4°	15.9°	14°	11.3°
Wire mesh mattress on geo-textile filter mat ($\varepsilon_s = 20°$)		—	0.385	0.599	0.707	0.820
Wire mesh mattress on granular filter ($\varepsilon_s = 25°$)		0.477	0.665	0.761	0.820	0.886

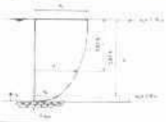
2) In case of very high design flow velocities ($u_b > 3$ m/s) or large wave heights ($H > 1$m) a granular sub-layer with minimum thickness of 0.2m should be provided between geo-textile filter mat and wire mesh mattress.

Specifications and Construction Details

Wire material and anchoring:
Besides the sufficient weight of the mattress a proper interlocking between the individual mattresses and appropriate anchoring of the mattress elements is most important. The diameter of the wire material should 4mm minimum, the anchor and interconnecting cables should be chosen to 10mm (strand-wire). In case wire mesh mattress systems are applied as a launching apron only proprietary box gabions (e.g. RENO©) should be used.

Brick filled mattress systems

Design against current



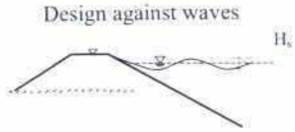
Minimum thickness of mattress $D_n \geq \frac{\bar{u}^2}{2g} \cdot \frac{0.035}{\psi_{cr}} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{(1-n) \cdot \Delta_m \cdot K_S}$

D_n	[m]	thickness of mattress system
$\bar{u}$	[m/s]	depth averaged flow velocity if replaced by $u_b = 0.6\ \bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
n	[-]	volume of voids in the mattress fill (approx. n = 0.15)
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
g	[m/s²]	acceleration due to gravity (= 9.81)
ϕ_{SC}	[-]	stability factor for current
ψ_{cr}	[-]	critical shear stress parameter
K_t	[-]	Turbulence factor
K_h	[-]	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2 \cdot \left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	[-]	bank normal slope factor $K_S = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for B.D. rivers and a conservative assumption [1]
α	[°]	slope angle of bank or structure
ε_s	[°]	angle of repose considering the material specific internal friction

Coefficient	Unit	Value	Coefficient	Unit	Value
Turbulence intensity K_t			**Stability factor ϕ_{SC}**		
Normal turbulence in rivers	[-]	1.0	Continuous Protection	[-]	0.50
Non-uniform flow with increased turbulence, mild outer bends	[-]	1.5	Exposed edges, transitions	[-]	1.00
High turbulence, local disturbances, sharp outer bends	[-]	2.0			
Angle of repose ε_s [2]			**Critical shear stress parameter ψ_{cr} (Shields)**	[-]	0.07
Geo-textile	[°]	20			
Granular	[°]	25			
Material density ρ_s					
Brick fill	[kg/m³]	1800			

Brick filled mattress systems

Design against waves



Minimum thickness of mattress $D_n \geq \dfrac{H_S \cdot \xi_z^b}{(1-n) \cdot \Delta_m \cdot \psi_u \cdot \phi_{SW} \cdot \cos\alpha}$; $(\xi_z < 3, \cot\alpha \geq 2)$

D_n	[m]	thickness of mattress system
H_s	[m]	significant wave height
n	[-]	volume of voids in the mattress fill (approx. n = 0.15)
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
ϕ_{SW}	[-]	stability factor for wave loads
ψ_u	[-]	system specific stability upgrading factor
ξ_z	[-]	wave similarity parameter $\tan\alpha \cdot \dfrac{1.25 \cdot T_m}{\sqrt{H_S}}$
α	[°]	slope angle of bank or structure
T_m	[s]	mean wave period
b	[-]	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

Coefficient	Unit	Value
Stability factor for incipient motion ϕ_{SW}	[-]	2.25
Stability upgrading factor ψ_u	[-]	2.50
Interaction Coefficient b	[-]	0.50

U 80

Brick filled mattress systems

Supplementary Information/
Specifications/ Construction Details

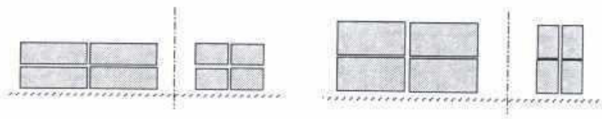
Supplementary Information

[1] slope factor K_s

Embankment						
	slope	1 : 2.5	1 : 3	1 : 3.5	1 : 4	1 : 5
	angle	21.8°	18.4°	15.9°	14°	11.3°
Wire mesh mattress on geo-textile filter mat ($\varepsilon_s = 20°$)		—	0.385	0.599	0.707	0.820
Wire mesh mattress on granular filter ($\varepsilon_s = 25°$)		0.477	0.665	0.761	0.820	0.886

[2] In case of very high design flow velocities ($u_b > 3$ m/s) or large wave heights ($H > 1$m) a granular sub-layer with minimum thickness of 0.2m should be provided between geo-textile filter mat and wire mesh mattress.

A minimum number of two layers must be considered for brick mattresses. As an approximation for Structure Category 2 (SC2) and 3 (SC3) the following arrangements are suggested:



Specifications and Construction Details

Wire material and anchoring:
Besides the sufficient weight of the mattress a proper interlocking between the individual mattresses and appropriate anchoring of the mattress elements is most important. The diameter of the wire material should 4mm minimum, the anchor and interconnecting cables should be chosen to 10mm (strand-wire). Brick filled wire mesh mattress systems should not be applied as a launching apron.

Cable Connected Concrete Blocks

(Articulating mattress)

Design against current

Minimum thickness $D_n \geq \frac{\bar{u}^2}{2g} \cdot \frac{0.035}{\psi_{cr}} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{(1-n) \cdot \Delta_m \cdot K_S}$

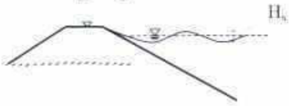
D_n	[m]	nominal thickness of mattress - system (height of concrete units)
$\bar{u}$	[m/s]	depth averaged flow velocity if replaced by $u_b = 0.6\ \bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
n	[-]	volume of voids between blocks per unit area (typical range: $0.1 \leq n \leq 0.3$)
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
g	[m/s²]	acceleration due to gravity (= 9.81)
ϕ_{SC}	[-]	stability factor for current
ψ_{cr}	[-]	critical shear stress parameter
K_t	[-]	turbulence factor
K_h	[-]	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2 \cdot \left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	[-]	bank normal slope factor $K_S = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for B.D. rivers and a conservative assumption [1)]
α	[°]	slope angle of bank or structure
ε_s	[°]	angle of repose considering the material specific internal friction

Coefficient	Unit	Value	Coefficient	Unit	Value
Turbulence intensity K_t			**Stability factor ϕ_{SC}**		
Normal turbulence in rivers	[-]	1.0	Continuous Protection	[-]	0.50
Non-uniform flow with increased turbulence, mild outer bends	[-]	1.5	Exposed edges, transitions	[-]	1.10
High turbulence, local disturbances, sharp outer bends	[-]	2.0			
Angle of repose ε_s			**Critical shear stress parameter ψ_{cr} (Shields)**	[-]	0.06
Geo-textile	[°]	20			
Granular	[°]	25			
Material density ρ					
Concrete ρ_c	[kg/m³]	2000			

Cable Connected Concrete Blocks

(Articulating mattress)

Design against waves



Minimum thickness $D_n \geq \dfrac{H_S \cdot \xi_z^b}{(1-n)\cdot \Delta_m \cdot \psi_u \cdot \phi_{Sw} \cdot \cos\alpha}$; ($\xi_z < 3$, $\cot\alpha \geq 2$)

D_n	[m]	nominal thickness of mattress - system (height of concrete units)
H_s	[m]	significant wave height
n	[-]	volume of voids between blocks per unit area (typical range: $0.1 \leq n \leq 0.3$)
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w)/\rho_w$
ϕ_{SW}	[-]	stability factor for wave loads
ψ_u	[-]	system specific stability upgrading factor
ξ_z	[-]	wave similarity parameter $\tan\alpha \cdot \dfrac{1.25 \cdot T_m}{\sqrt{H_S}}$
α	[°]	slope angle of bank or structure
T_m	[s]	mean wave period
b	[-]	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

Coefficient	Unit	Value
Stability factor for incipient motion ϕ_{SW}	[-]	2.25
Stability upgrading factor ψ_u [3)]	[-]	1.8
Interaction Coefficient b	[-]	0.67

[3)] for blocks only connected to a geo-textile the upgrading factor must be reduced to $\psi_u = 1.5$

Cod6

Cable Connected Concrete Blocks

(Articulating mattress)

Supplementary Information/ Specifications/ Construction Details

Supplementary Information

[1] slope factor K_s

Embankment	Slope	1 : 2.5	1 : 3	1 : 3.5	1 : 4	1 : 5
	Angle	21.8°	18.4°	15.9°	14°	11.3°
Rip-rap on geo-textile filter mat ($\varepsilon_s = 20°$)		—	0.385	0.599	0.707	0.820
Rip-rap on granular filter ($\varepsilon_s = 25°$)		0.477	0.665	0.761	0.820	0.886

Specifications and Construction Details

Casting of the concrete units will be carried out on site. Before concreting the cross-wise cable connections (strand wire) must be placed in position. To enhance the release of the concrete elements the form work should be conical.

The cables normal to the structure crest must be sufficiently anchored in a trench at the berm in front of the upper revetment (compare also figure at the top of this pages, the river is on the right hand side). Steel piles of 80mm diameter, reinforced by additional flanges to increase the pile resistance, with a minimum embedment length of 4m should be utilized for this purpose (see figure below left). After piling the trench must be filled by adequate material like boulders or rip-rap (see figure below right, filling incomplete).

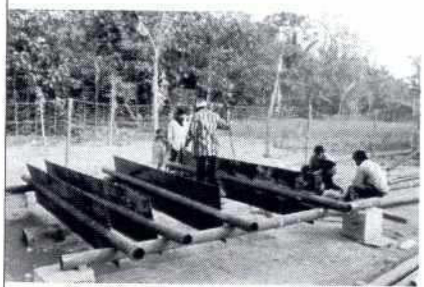
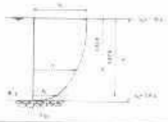


Figure A: Anchor piles with flanges

Figure B: Anchor trench partly filled

Cube-shaped Concrete Blocks
(Randomly dumped in multi-layer)

Design against current



Minimum Size[1] $D_n \geq \frac{\bar{u}^2}{2g} \cdot \frac{0.035}{\psi_{cr}} \cdot \frac{\phi_{SC} \cdot K_t \cdot K_h}{\Delta_m \cdot K_S}$

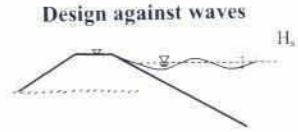
D_n	[m]	size (arris length) of single cc-block
$\bar{u}$	[m/s]	depth averaged flow velocity if replaced by $u_b = 0.6\,\bar{u}$ (theoretical bottom flow velocity for a logarithmic velocity profile) a value of $K_h = 1.0$ must be applied
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
g	[m/s²]	acceleration due to gravity (= 9.81)
ϕ_{SC}	[-]	stability factor for current
ψ_{cr}	[-]	critical shear stress parameter
K_t	[-]	turbulence factor
K_h	[-]	depth factor, dependent on the assumed velocity profile and the water depth to equivalent roughness height ratio: $K_h = 2 \cdot \left[\log\left(\frac{12h}{k_r}\right)\right]^{-2}$ with $k_r = D_n$ for relatively smooth material
K_s	[-]	bank normal slope factor $K_S = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\varepsilon_S}\right)^2}$ neglecting the longitudinal slope of the bank or structure, which is reasonable for B.D. rivers and a conservative assumption [2]
α	[°]	slope angle of bank or structure
ε_s	[°]	angle of repose considering the material specific internal friction

Coefficient	Unit	Value	Coefficient	Unit	Value
Turbulence intensity K_t			**Stability factor ϕ_{SC}**		
Normal turbulence in rivers	[-]	1.0	Continuous Protection	[-]	0.75
Non-uniform flow with increased turbulence, mild outer bends	[-]	1.5	Exposed edges, transitions	[-]	1.25
High turbulence, local disturbances, sharp outer bends	[-]	2.0			
Angle of repose ε_s	[°]	40	**Critical shear stress parameter ψ_{cr} (Shields)**	[-]	0.035
Material density ρ					
Concrete (khoa aggregates) ρ_c	[kg/m³]	2000			
Water ρ_w	[kg/m³]	1000			

Cube-shaped Concrete Blocks

(Randomly dumped in multi-layer)

Design against waves



Minimum Size[1)] $D_n \geq \dfrac{H_S \cdot \xi_z^b}{\Delta_m \cdot \psi_u \cdot \phi_{SW} \cdot \cos\alpha}$ ($\xi_z < 3$, $\cot\alpha \geq 2$)

D_n	[m]	size (arris length) of single cc-block
H_s	[m]	significant wave height
Δ_m	[-]	relative density of submerged material $(\rho_s - \rho_w) / \rho_w$
ϕ_{SW}	[-]	stability factor for wave loads
ψ_u	[-]	system specific stability upgrading factor
ξ_z	[-]	wave similarity parameter $\tan\alpha \cdot \dfrac{1.25 \cdot T_m}{\sqrt{H_S}}$
α	[°]	slope angle of bank or structure
T_m	[s]	mean wave period
B	[-]	wave-structure interaction coefficient, mainly dependent on roughness and porosity of protective material

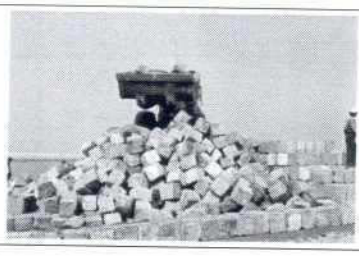
Coefficient	Unit	Value
Stability factor for incipient motion ϕ_{SW}	[-]	2.25
Stability upgrading factor ψ_u	[-]	1.50
Interaction Coefficient b	[-]	0.67

General remarks:

Following the layout of the standardized revetments, the base level of the falling apron is a few decimetres above FPL, therefore prevailing wave heights are rather limited due to the restricted fetch length as compared to high water stages. In addition, the actual wave loads are reduced, because the outer slope of the rubble structure is quite steep (cot α = 2), so that a large part of the energy is reflected. Thus, normally wave impacts are not decisive for the design of falling aprons. Nevertheless, excessive rocking of the individual concrete units should be avoided. The minimum required dimensions (wave impact) should be compared with results obtained from design against current loads and should be judged on the background of storm frequencies, exposure to wave action, etc.

Cube-shaped Concrete Blocks
(Randomly dumped in multi-layer)

Effective Volume and Dimensions



Required volume of scour protection blanket per metre bank line:

$$V_{FA} = 1.5 \cdot D_n \cdot \sqrt{5} \cdot y_{BL} \cdot C_{FA}$$

with

V_{FA}	(m^3/m)	Volume of falling apron per linear metre protected length
D_n	(m)	block size (1.5 D_n is the proposed layer thickness after scouring without voids)
y_{BL}	(m)	vertical distance between base level of falling apron at time of construction and deepest point of the expected design scour hole
C_{FA}	[-]	flow attack coefficient: 1.50 (moderate flow attack) 1.75 (strong flow attack)

Width of the falling apron:

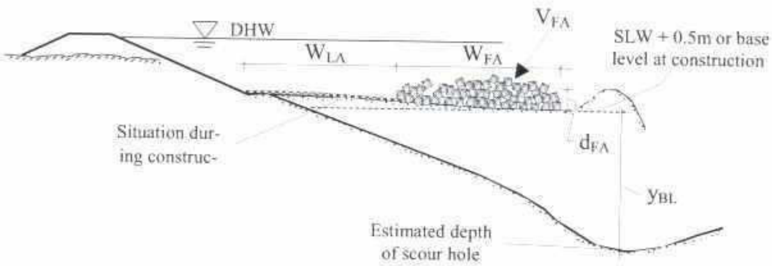
$$W_{FA} = 0.7 \text{ to } 1.0 \cdot y_{BL}$$

Theoretical thickness of the falling apron (without voids):

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

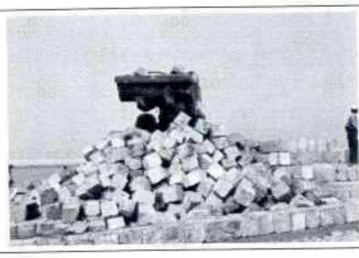
The actual thickness of the falling apron material during construction is larger, because of the voids between the individual CC-blocks. The minimum thickness of the falling apron should be larger than $d_{FA} = 3\ D_n$, whereas the maximum thickness is dependent on the construction technique used (manual labour or mechanically placed) and the maximum tolerable load on the existing sub-soil. To control the quantity of material to be provided for the designed falling apron width W_{FA}, the number of blocks per linear metre of protected length can be calculated by

$$n = \frac{V_{FA}}{D_n^3}$$



Cube-shaped Concrete Blocks
(Randomly dumped in multi-layer)

Supplementary Information



Supplementary Information

[1] It is recommended to apply a minimum block size $D_n = 0.3$m.

[2] slope factor K_s

Due to the fact it is implicitly wanted, that individual blocks start to proceed down the developing scour hole in front of the falling apron, normally no sub-layer is required. The driving force is mainly gravity induced. After stopping of further erosion due to sufficient coverage of the scour hole, the protective concrete units must remain stable under the prevailing current. To consider this (final) design condition a slope factor of $K_s = 0.75$ is recommended to be used for the design of the falling apron.

Specifications and Construction Details

It is recommended to dump the blocks to form a mound, rather than to place the elements in rows or columns, because it is assumed that the articulation of the falling apron is improved.

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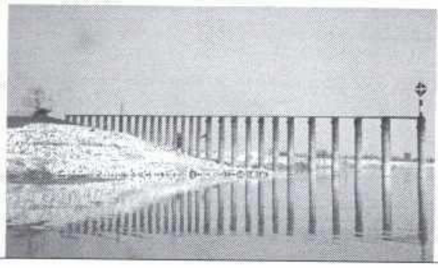
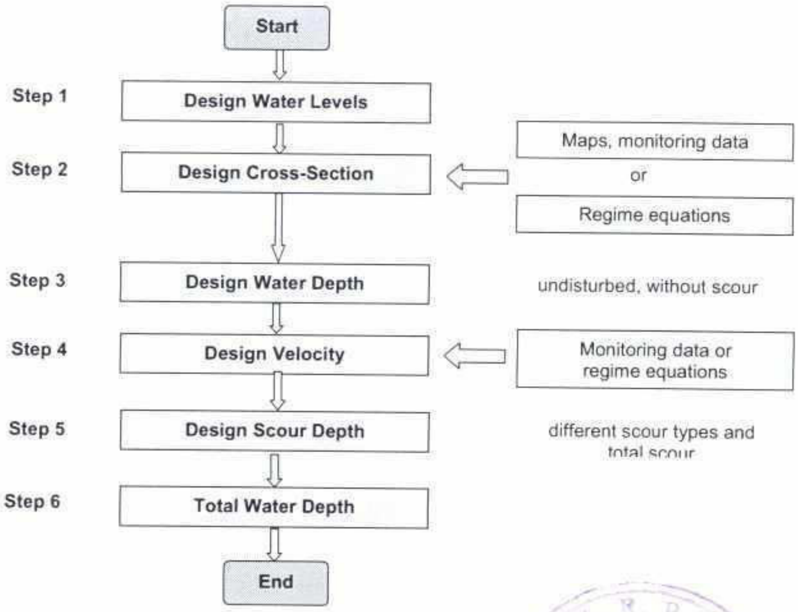
Permeable Groynes

Estimate of scour depth and total water depth at outer groyne end

h_0 ; $y_{s,0}$

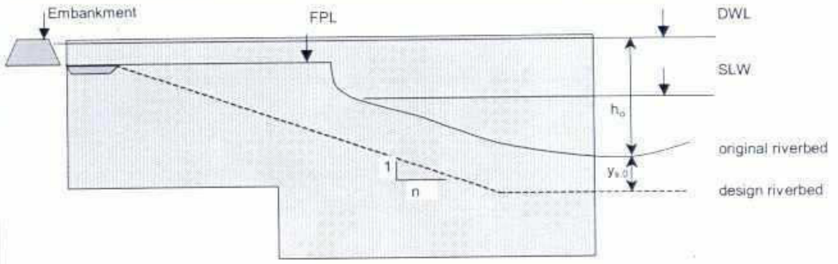


For standardized groynes, the design scour depth is chosen according to the Structure Class. If a classification is not clearly possible, the scour depth at the outer end of the groyne can be estimated according to the following procedure.



Design Sketch

Ref. to GUIDE



Step 1: Design Water Levels

					Ref. to GUIDE
Standard Low Water	SLW	0,00	m SLW	*reference water level*	
Design Water Level	DWL	+ 7,70	m SLW	*25 years return period*	Sec. 4.2.3
Flood plain level	FPL	+ 4,80	m SLW		

Step 2: Design Cross-Section

					Ref. to GUIDE
Cannel width at bankful discharge	B_{ch}	500	m	*influenced width*	Sec. 4.4.1 / 4.4.2 (a)
Channel Area	A_{ch}	4350	m	*related to DWL*	Sec. 4.4.1
Averaged water depth of channel	h_{ch}	8,70	m	*related to DWL*	Eq. 4.4-4
Relative bend radius	r_c/B_{ch}	2,50	-	*assume 2.5 as first approach*	Eq. 4.4-5

Step 3: Design Water Depth

					Ref. to GUIDE
Water depth at thalweg of river bend	h_0	18,01	m	*related to DWL*	Eq. 4.4-6
	h'_0	15,11	m	*related to FPL*	as Eq. 4.4-4
Water depth at outer end of groyne	h_1	18,01	m	*for river bends assume* $h_1 = h_0$	Sec. 4.4.5 (c)

Step 4: Design Velocity

					Ref. to GUIDE
Upstream depth averaged flow velocity	u_1	3,20	m/s	*from monitoring data or according to Structure Class*	Sec. 4.4.5 (c) Table 2.3-1

If no sufficient or reliable data is available, the design velocity may be estimated through regime equations as follows:

Channel discharge	Q_{ch}	9.675	m³/s	*from regime equations*	Eq. 4.4-3
Cross-sectional averaged flow velocity	u_{ch}	2,22	m/s		Eq. 4.5-1
Depth-averaged velocity in the thalweg	u_0	3,20	m/s		Eq. 4.5-2
Upstream depth averaged flow velocity	u_1	3,20	m/s	*for river bends assume* $u_1 = u_0$	Sec. 4.4.5 (c)
Critical velocity for sediment motion	u_{cr}	1,00	m/s	*shallw=0.4m/s; deep=1.0 m/s*	Sec. 4.5.5

Step 5: Design Scour

					Ref. to GUIDE
Empirical coefficient for permeability	K_{Sh}	1,80	$s^{0.67}/m^{0.33}$	*1.2 (80%) to 2.0 (50%)*	Tab. 4.4-1
Empirical coefficient for bed protection	K_{bp}	1,10	-	*non=1.0; with=1.1*	Tab. 4.4-1
Empirical coefficient for floating debris	K_{fd}	1,00	-	*non=1.0; <1m=1.2; >1m=1.3*	Tab. 4.4-1
Potrusion lenght of total groyne	b	150,00	m	*measured from bank line*	Fig. 4.4-6
Local scour depth	$y_{s;\,local}$	6,00	m		Eq. 4.4-12
Bend scour depth	$y_{s;\,bend}$	12,61	m		Eq. 4.4.-8
Confluence scour depth	$y_{s;\,conf}$	0,00	m		Eq. 4.4-10
Total design scour	$y_{s,0}$	9,72	m	*related to thalweg*	Eq. 4.4-14(a)

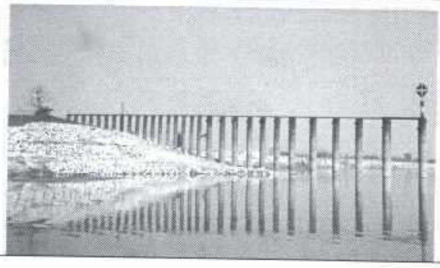
Step 6: Total Water Depth

					Ref. to GUIDE
Total water depth	$h_0 + y_{s,0}$	27,73	m	*related to DWL*	

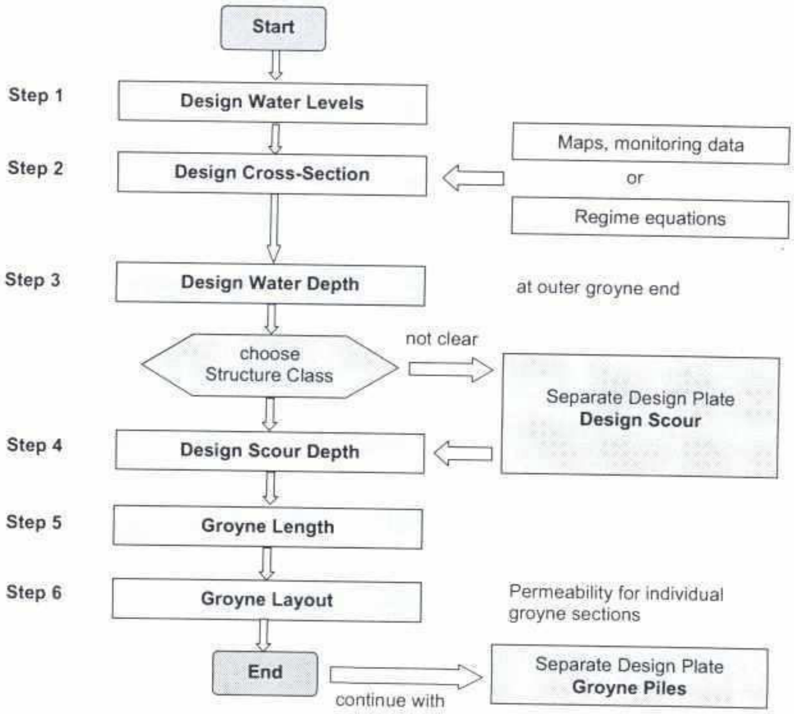
Permeable Groynes

Dimensioning of groyne length and permeability

L_G ; P



The dimensioning is being carried out in 6 steps, which are explained in detail on the following pages.



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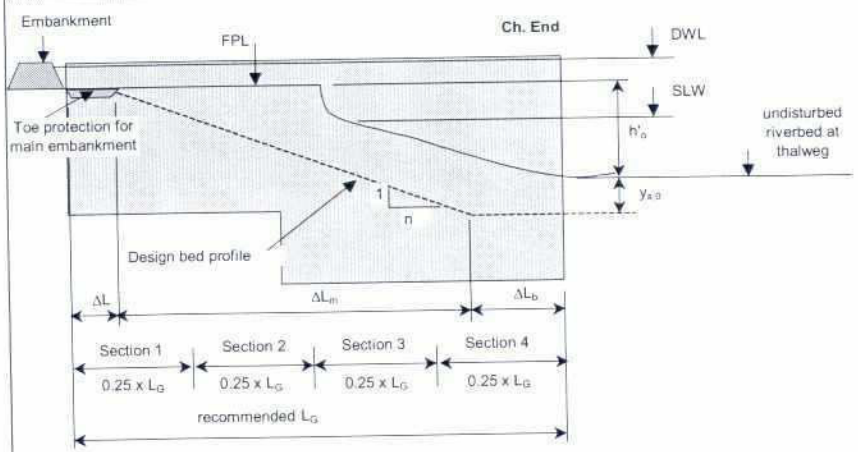
Design Sketch



Step 1: Design Water Levels

Ref. to GUIDELINES

Standard Low Water	SLW	0,00	m SLW	reference water level	
Design Water Level	DWL	+ 7,70	m SLW	25 years return period	Sec. 4.2.3
Flood plain level	FPL	+ 4,80	m SLW		

Step 2: Design Cross-Section

Ref. to GUIDELINES

Cannel width at bankful discharge	B_{ch}	500	m	influenced width	Sec. 4.4.1 / 4.4.2 (a)
Channel Area	A_{ch}	4350	m	related to DWL	Sec. 4.4.1
Averaged water depth of channel	h_{ch}	8,70	m	related to DWL	Eq. 4.4-4
Relative bend radius	r_c/B_{ch}	2,50	-	assume 2.5 as first approach	Eq. 4.4-5

Step 3: Design Water Depth

Ref. to GUIDELINES

Water depth at thalweg of river bend	h_0	18,01	m	related to DWL	Eq. 4.4-6
	h'_0	15,11	m	related to FPL	as Eq. 4.4-4

Step 4: Design Scour Depth

Ref. to GUIDELINES

Total water depth	$h_0+y_{s,0}$	25,00	m	related to DWL according to Structure Class (SC) or estimate according to separate design plate "Design Scour Depth"	Table 2.3-1
Total design scour depth	$y_{s,0}$	6,99	m	>= 0m	

If a classification is not clearly possible, the scour depth can be estimated according to Eq. 4.4-12.
A separate design plate has been prepared for this.

Coo2

Step 5: Groyne Length

Ref. to GUIDELINES

Safety margin to embankment toe	ΔL	20,00	m	*min 10 m*	*Sec. 6.2.1*
Bed slope towards bank	n	5,00	-	*n = 5 to 7 for sandy bed*	*Sec. 6.2.1*
Angle of oblique flow approach	Θ	30,00	degree	*15° to 45°*	*Sec. 6.2.1*
Angle of flow separation	β	15,00	degree	*estimate 15° and approve later*	*(Eq. 6.2-4)*
Length of local scour	l_s	27,96	m	$= 4 * y_{s,0}$	*Eq. 6.2-1, remarks*
Length of assumed natural slope	ΔL_m	111,00	m	$= n * (h'_o + y_{s,0})$	*see Fig. above*
Safety margin for oblique flow attaque	ΔL_b	14,00	m	$= 1/2\, l_s * \tan(\beta + \Theta)$	*see Fig. above*
Total length of groyne recommended	L_G	145,00	m	$= \Delta L + \Delta L_m + \Delta L_b$	*Eq. 6.2-1*
max	L_G	200,00	m	$= 0.4 * B_{ch}$	*Eq. 6.2-2*
design length	L_G	145,00	m	*choose amongst recommended L_G and max L_G*	

Step 6: Groyne Layout

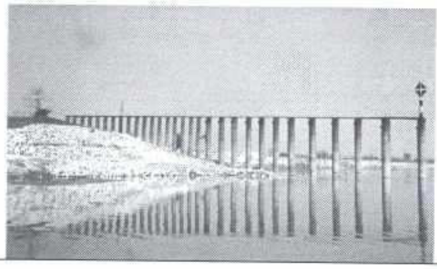
Ref. to GUIDELINES

Chainage	Section Number	Section Length	Permeability	
[-]	[-]	[m]	[%]	
Ch. 0.00				
0.00 to 0.25 L_G	1	36,25	50%	*Table 6.1-2*
0.25 to 0.50 L_G	2	36,25	60%	*Table 6.1-2*
0.50 to 0.75 L_G	3	36,25	70%	*Table 6.1-2*
0.75 to 1.00 L_G	4	36,25	80%	*Table 6.1-2*
Ch. End				
Total length:		**145,00 m**		

Permeable Groynes

Estimate of embedment length and pile dimensions

t_e ; D; t; μ



The pile design comprises the calculation of the required **embedment length t_e** for the individual piles as well as the determination of the main pile dimensions, that is the **pile diameter D**, the **wall thickness t** (for steel piles) and the **reinforcement content μ** (for concrete piles).

The embedment length is calculated according to a modified approach of BLUM taking into account the effect of spatial earth pressure. This approach requires intensive computations, wherefore design graphs have been prepared, which can be used for the preliminary pile design. These graphs are presented at the end of this plate. Beside the embedment length, also the main pile dimensions can be read from these graphs.

In order to ease the pile design, the following **assumptions** have been made for the elaboration of the design graphs:

A design wave height of 1.0 m has been assumed, corresponding to the observed wave heights at the Jamuna (wave period = 3.5 s). Similarly, a 1m thick layer of floating debris was assumed. Both loads have been evaluated, but only the more unfavourable one entered into the design graphs.

No ship impact and no earthquake loads were considered, as these are exceptional loads which are contradictory to the idea of standardized structures, where certain damages to the structures under very unfavourable conditions are tolerable with regard to an economic design. To prevent ship impact, the outer groyne piles should be marked with a navigation sign.

The permeability of the groyne was uniformly set to 50%, corresponding to the minimum recommended value.

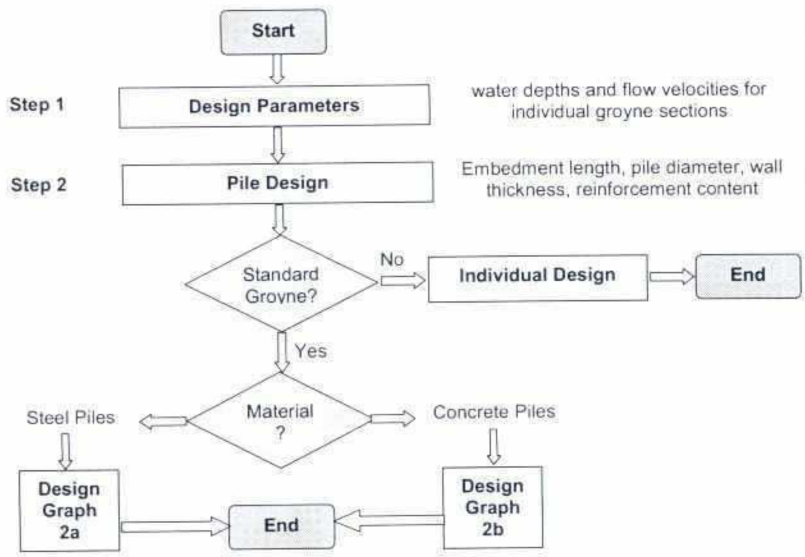
A sandy, non-cohesive soil with typical characteristics of the Jamuna river bed was assumed (γ'=10kN/m³, φ'=35° and c_u=0kN/m²).

The calculation of the embedment length l_E was done according to EAU 1990, restricting the minimum embedment length to t_e = 5,0 m.

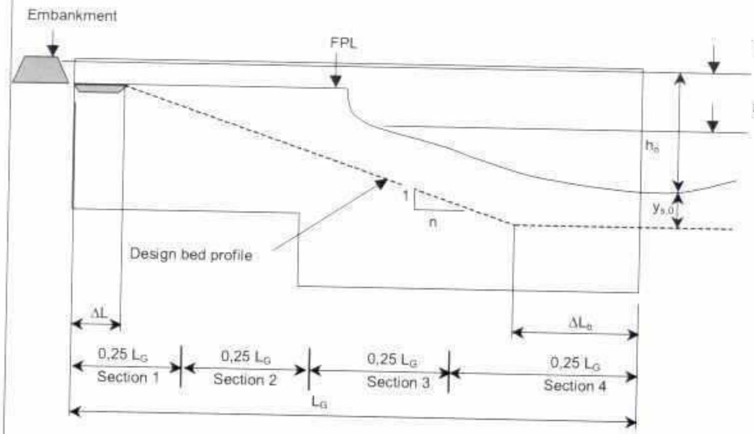
The **steel pile** dimensions have been computed on the basis of a steel grade S 235 JR (yield stress 240 N/mm²). To prevent buckling of the steel piles and stability problems during pile installation, the wall thickness of the tubular steel piles has been set to be at least 1/100 of the respective pile diameter.

The design of **concrete piles** was done according to Eurocode 2 with safety factor $\psi=1.0$ for the loads. Concrete Class B35, DIN 1045, has to be specified for pile construction. However, for design purpose only Class B 25 with a strength of f_{ck} = 25 N/mm² has been considered in the calculations to give consideration to executional deficiencies during in-situ pile construction below ground water level. A reinforcing steel Grade 60 ($f_{y,k}$ = 414 N/mm²) has been considered for the design graphs. The maximum percentage of reinforcement should not exceed about 2 % of the pile's cross-section in order to guarantee sufficient flow of concrete around the steel bars during concreting.

The main steps of the pile design procedure are shown in the following graph. The single steps are explained in detail on the following page. The design graphs for steel and concrete piles are presented at the end of this plate.



Design Sketch



Step 1: Design Water Level

Ref. to GUIDELINES

Design Water Level DWL + 7,70 m SLW *25 years return period* *Sec. 4.2.3*

Step 2: Design Water Depth

Ref. to GUIDELINES

Total water depth at outer groyne pile $h_0+y_{s,0}$ 25,00 m *related to DWL*
according to Structure Class (SC) *Table 2.3-1*
or from design plate "Design Scour Depth"

Step 3: Design Velocity

Ref. to GUIDELINES

Upstream depth averaged flow velocity u_1 2,00 m/s *at the outer groyne pile* *Sec. 4.4.5 (c)*
obtain from monitoring data
or according to Structure Class (SC) *Table 2.3-1*
or from design plate "Design Scour Depth"

Step 4: Design Parameters for Individual Groyne Sections

Ref. to GUIDELINES

Groyne Section Chainage	Water Depth Upstream Water Depth for Piles according to Design Bed Profile related to DWL	Flow Velocity Reduction Factor	Upstream Velocity
[-]	[m]	[-]	[m/s]
Tab 6.1-2	*Fig. 6.2-1*	*Fig. 6.3-2*	*= u₁ x Factor*
Ch. 0.00			
0.00 to 0.25 L_G	6,15	0,50	1,00
0.25 to 0.50 L_G	13,40	0,75	1,50
0.50 to 0.75 L_G	20,65	1,00	2,00
0.75 to 1.00 L_G	25,00	1,00	2,00
Ch. End			

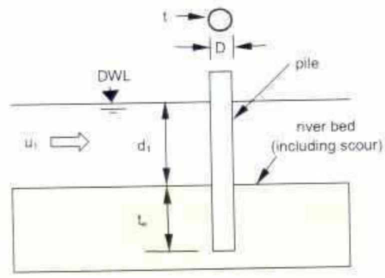
Step 2a: Design of Steel Piles

Ref. to GUIDELINES

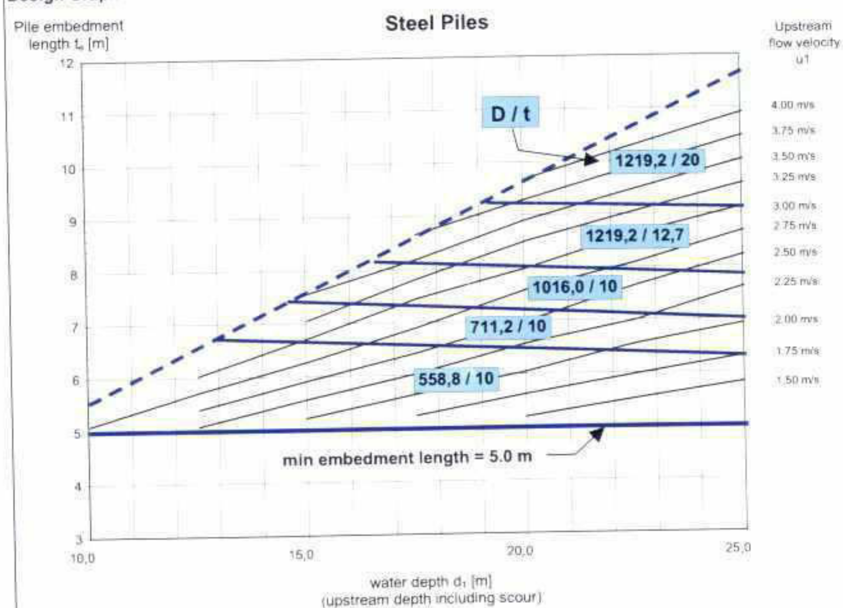
Design Assumptions for Design Graph

			Value	Unit	Ref.
Design Wave	Height	H	1,00	m	Section 6.3.2 + 4.2.4
	Period	T	3,50	s	
Floating Debris	Thickness	h_d	1,00	m	Section 6.3.3 + 4.6.1
Ship Impact			not considered		Section 4.6.2
Earthquake			not considered		Section 4.6.3
Groyne Permeability		P	50	%	
Soil Characteristics	Weight	γ'	10,00	kN/m³	
	Friction Angle	φ'	35	°	
	Cohesion	c'	0	kN/m²	
Equivalent Roughness of Bed Material		ks	0,10	m	Section 6.3.1
Steel Grade			S 235 JR		

Definition Sketch



Design Graph



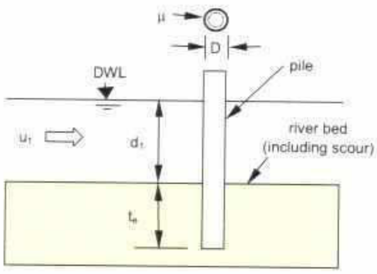
Step 2b: Design of Concrete Piles

Ref. to GUIDELINES

Design Assumptions for Design Graph

Design Wave	Height	H	1,00	m	Section 6.3.2 + 4.2.4
	Period	T	3,50	s	
Floating Debris	Thickness	h_o	1,00	m	Section 6.3.3 + 4.6.1
Ship Impact			not considered		Section 4.6.2
Earthquake			not considered		Section 4.6.3
Groyne Permeability		P	50	%	
Soil Characteristics	Weight	γ'	10,00	kN/m³	
	Friction Angle	φ'	35	°	
	Cohesion	c'	0	kN/m²	
Equivalent Roughness of Bed Material		ks	0,10	m	Section 6.3.1
Concrete Grade			C 20/25		
Reinforcement Steel Grade			500 S		

Definition Sketch



Design Graph