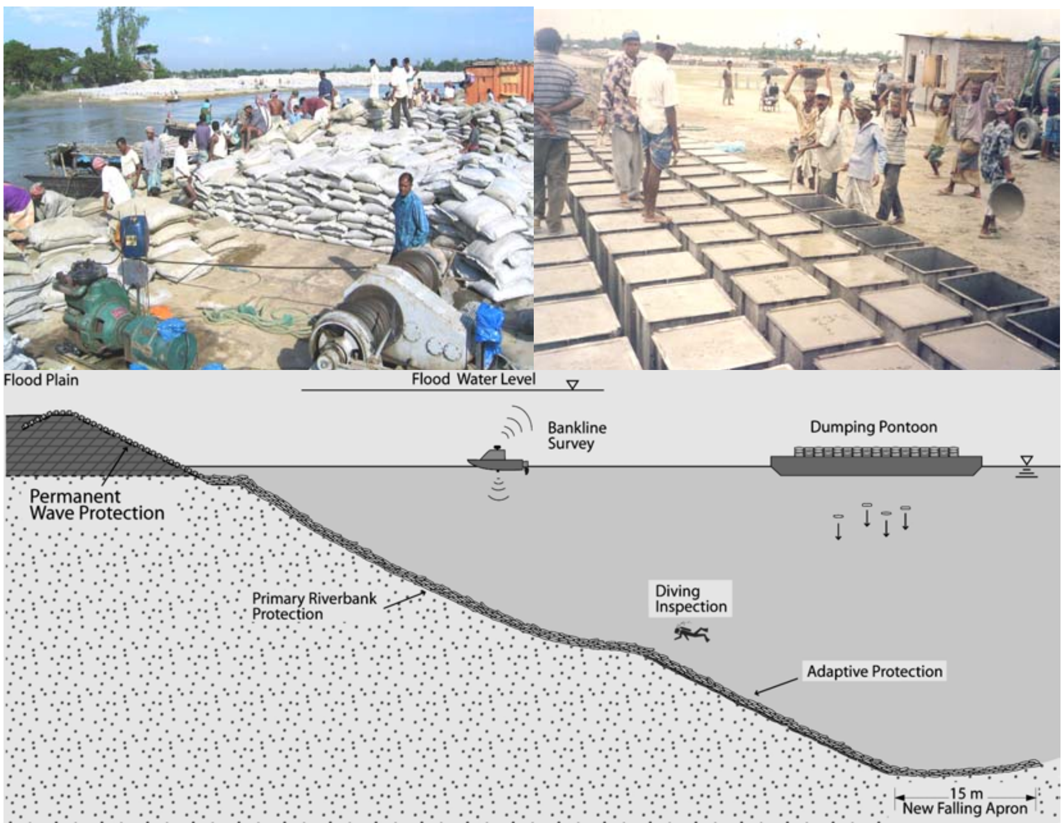
Government of the Peoples' Republic of Bangladesh

Asian Development Bank

Ministry of Water Resources

Bangladesh Water Development Board

Jamuna-Meghna River Erosion Mitigation Project (JMREMP)



Guidelines for River Bank Protection

Bureau of Research Testing and Consultancy
Bangladesh University of Engineering & Technology

May 2008

PREFACE

In 1993 BWDB published the first Design guidelines on Guide to Planning and Design of River Training and Bank Protection Works, Design Circle-II. This was followed by the FAP21/22 publication on Guidelines and Design Manual for Standardized Bank Protection Structures. Recently a guidelines for Design of River bank Protection and Manual has been Published in 2007 under JMREMP. Since the publication of earlier two guidelines, there have been quite few advances in understanding of river bank Protection materials and of the design and behaviors of the river training works in the river environment. The publication under JMREMP has reviewed not only the earlier guidelines but also introduces on the use of geobags as new construction materials and also a concept of adaptive approach in the constructions and strengthening of the river bank protection works. The present guideline considers all the previous publications and collates available research data, technical information together with practical experience gained by practitioners and designers of BWDB. In doing so, care has been taken to provide a balance between traditional design practices and the new concepts of adaptive approaches with their limitations and extent to which engineering judgment are involved.

The guideline has a broad base content with general information on river behaviour, hydraulics, hydrology, river erosion management planning and strategy etc in part-I, Design related relevant information in part-II and Constructional aspects in part-III. Therefore the audience of the guidelines is wide and includes planners, developers, engineering consultants & designers and the like. The manual assumes that the reader has a level of technical knowledge typically corresponding to a minimum of a degree in Civil/Water Resources Engineering with some years of experience or equivalent qualifications and experience. The manual addresses the need of a range of people involved in the river bark protection works in Bangladesh.

The guidelines provide required information for design of the main river bank protection structures in Bangladesh. However the knowledge and experience from the guidelines can be utilized judiciously to design of river bank protection works in other rivers of Bangladesh given due judgment to that particular river environment.

ACKNOWLEDGEMENT

This “Guidelines for River Bank Protection” under JMREMP” has been prepared under a contract between BRTC, BUET and BWDB. Asian Development Bank financed the work under JMREMP project. The team of experts of BRTC expresses their deepest gratitude and appreciation to ADB for financial support and strong initiatives for the work. The team is very much grateful to Mr. Md. Makbul Hossain, Project Director of JMREMP and Mr. Knut Oberhagemann, Team Leader of JMREMP for their enthusiastic and continuous support to the work.

The team is also thankful to the Engineers of Design Circles of BWDB for their active interests and participation, constructive comments and valuable suggestions at all stages of the work. Suggestions and advice of other scientists in many technical issues encourages the consultants' team and enriches the work. In this respect the team would like to express their thanks to the participants of the Workshop in which the Draft Report of the Guidelines were presented and valuable suggestions and comments were received especially from DG, BWDB; ADG (O&M-II), ADG (Planning), BWDB and other concerned officials including Design Circles of BWDB and relevant water sector Organizations who are involved in studies and research on River Bank Protection Works in Bangladesh. Deep appreciations and gratefulness are expressed to Mr. Mukhles uz Zaman, former DG, BWDB for his constant inspiration, constructive comments and criticism.

The consultants' team hopes that the work contributes a small step towards systematic design and improved implementation procedure towards a safe, economic, environmental friendly bank protection structures for the major rivers of Bangladesh.

Prof. Dr. M. Monowar Hossain
Team Leader, BUET Team.

Table of Contents

Glossary

List of symbols

PART–I : General

CHAPTER ONE: Introduction

1.1	Importance	1
1.2	Design Manuals in Bangladesh	1
1.3	Scope of the Guideline and Manual	2
1.4	Layout of the Manual	2
1.5	The Manual	2

CHAPTER TWO: River System and Estimation of Design Water Level and Discharge

2.1	Alluvial Rivers	3
	2.1.1 Aggrading Rivers	4
	2.1.2 Degrading Rivers	4
	2.1.3 Meandering Rivers	4
	2.1.3.1 Causes of Meandering	5
	2.1.3.2 Parameters of Meandering	6
	2.1.4 Braiding and Anabranching Rivers	6
2.2	Rivers of Bangladesh	7
	2.2.1 Main Rivers and Hydrological Aspects	7
	2.2.2 River Network and Morphology	13
	2.2.3 Sediment Transport	14
2.3	Estimation of Design Discharge and Water Level	16
2.4	Frequency Analysis	16
	2.4.1 Analytical Frequency Analysis	16
	2.4.2 Graphical Frequency Analysis	21
2.5	Goodness-of-fit Tests	22
2.6	Testing for Outliers	24
2.7	Reliability of Analysis	25
2.8	Transfer of Discharge and Water Level	27
2.9	Bankful Discharge	27
2.10	Dominant Discharge Analysis	27

CHAPTER THREE: Bank Erosion and General Protection Measures

3.1	Introduction	39
3.2	River Instability and Bank Erosion	39
3.3	Mechanism and Processes of Bank Erosion	39
	3.3.1 Surface Erosion of Banks	39
	3.3.2 Mass Failure of Banks	41
3.4	Assessment of Erosion Potential	46
3.5	General Methods of Bank Protection	46
	3.5.1 General Concept	46
	3.5.2 Traditional and Low Cost River Training Measures	46
	3.5.3 Biotechnical River Bank Protection	51
	3.5.4 Standard Practices	52
	3.5.4.1 Groynes	52
	3.5.4.2 Revetments	55
	3.5.5 Criteria for Selection of Alternative Structure	59
3.6	Soil, Water, Structure Interaction	59
	3.6.1 Common Causes of Failure of Bank Protection	60
	3.6.2 Revetment Failure	61
	3.6.3 Groyne Failure	63
	3.6.4 Riprap Failure Modes	66
3.7	Remarks	67

CHAPTER FOUR: Geological, Seismic and Geotechnical Aspects

4.1	Geological Aspects	68
4.2	Seismic Aspects	71
4.3	Geotechnical Aspects	74
4.4	Geotechnical Boundary Conditions and Data Collection	74
4.5	Soil-investigation Programme	74
	4.5.1 Desk Studies	75
	4.5.2 Preliminary Soil Investigations	76
	4.5.3 Detailed Soil Investigations	76
	4.5.4 Laboratory Tests	78
4.6	Geotechnical Limit States for a Slope	79
	4.6.1 Macro Stability of Slope	80
	4.6.2 Phenomena Governing Macro Stability of Slope	80
	4.6.3 Slope Stability Analysis	81
	4.6.3.1 Swedish Method of Slices	81
	4.6.3.2 Simplified Bishop Method of Sliuces	83
	4.6.3.3 Translatinal Failure Analysis (for Stratified Soil)	85
	4.6.4 Safety Factor for Slope Stability Analysis	90
4.7	Micro Stability	91
4.8	Liquefaction	92
	4.8.1 Liquefaction Potential	92
4.9	Filters	94
4.10	Soil Characteristics of the Jamuna River/ Meghna River	96
	4.10.1 Flow Slides	97
4.11	Considered Slope	97

CHAPTER FIVE: General Planning of Riverbank Erosion

5.1 Riverbank Erosion Management 101
5.2 Design Principles 102
5.2.1 Phased Implementation Concept 103
5.2.2 Erosion Risk and Duration 103
5.3 Planning and Design 106
5.3.1 Erosion and Morphology Prediction 106
5.3.2 Prioritization 106
5.3.3 Protection Length 107
5.3.4 Standardized Interventions 107
5.3.5 Planning and Funding 109
5.3.6 Stock Piling of Construction Materials 110
5.3.7 Environmental Management 110

PART-II : Design

CHAPTER SIX: Data Requirements

6.1 Hydrologic and Hydraulic Parameters 113
6.2 Morphological Parameters 113
6.3 Schematization of Cross-section 114
6.4 Survey 116

CHAPTER SEVEN: Design Considerations for Bank Protection Works

7.1 General Approach 119
7.2 Design Criteria 119
7.2.1 Water Levels 100
7.2.2 Flow Velocity 101
7.2.3 Design Discharge 102
7.2.4 Waves 103
7.2.4.1 Wave Analysis 104
7.2.4.2 Data for Calculating Wave Heights 107
7.2.4.3 Waves in Rivers 107
7.2.4.4 Wind and Wave Environment in Bangladesh 107
7.2.5 Scour 109
7.2.6 Remarks on Design Scour Depth 135

CHAPTER EIGHT: Design of Revetments

8.1 General aspects 137
8.2 Layout Consideration 137
8.2.1 Design 137
8.2.2 Length and Alignment 138
8.2.3 Cross Sectional Requirement 138
8.2.4 Erosion Resistance to Subsoil 139
8.2.5 Terminations and Transitions 141
8.3 Stability of Revetment under Current Attack 141
8.3.1 Loose Units 141
8.3.2 Stone Gabions and Mattresses 146
8.4 Stability of Revetment under Wave Attack 147
8.5 Thickness and Grading of Riprap 149
8.5.1 Riprap Thickness 149
8.5.2 Riprap Grading 150
8.6 Design of Filter Material 151
8.6.1 General 151
8.6.2 Granular Filter 152
8.6.3 Fiber Filter 153
8.7 Design of Toe Protection 158
8.7.1 Toe Scour Estimation and Protection 158
8.7.2 Toe Protection Methods of Revetment 158
8.7.3 Dimension of Falling Apron 160
8.7.4 Gradation of Launching Material 166
8.8 Innovative Revetments 167
8.8.1 General 167
8.8.2 General Stability Criteria 167
8.8.3 Properties of Geosynthetic Material/Geotextiles 168
8.8.4 Study f Geo-bag as an Alternative Launching Material 169
8.8.5 Observations made on use of geo-bags (PIRDP and MDIP) 173
8.8.6 Worked-out Design Example 174

CHAPTER NINE: Design of Groynes

9.1 General 175
9.2 Selection of type of Groyne 175
9.3 Length and Spacing of Groynes 175
9.4 Layout Alignments 176
9.5 Cross-Section Requirements 177
9.6 Design Considerations for R.C.C. Groyne 180
9.6.1 Introduction 180
9.6.2 Design Parameters 180
9.6.3 Length and spacing of the RCC Groyne 181
9.6.4 Design of Bed Protection 182
9.6.5 Typical design of a groyne in Ganges River 182

PART-III : Construction

CHAPTER TEN: Earth Works

10.1	General Description		187
	10.1.1	Excavation in Dry Condition	187
	10.1.2	Wet Excavation	187
	10.1.3	Compaction Equipment	188
10.2	Sample Specifications		189
	10.2.1	Clearing of Construction Site	190
	10.2.2	Excavation	190
	10.2.3	Filling Works	191

CHAPTER ELEVEN: Filters

11.1	General		195
11.2	Granular Filter		195
	11.2.1	General Description	195
	11.2.2	Materials for Granular Filter	195
	11.2.3	Fine Filter Material (sand)	196
	11.2.4	Coarse Filter Material (gravel, stone chips or khoa)	196
	11.2.5	Mixed Granular Filter	196
	11.2.6	Foundation Preparation and Laying Filter	196
11.3	Geotextile Filter		197
	11.3.1	General	197
	11.3.2	Geotextile used as Revetment Filter	197
	11.3.3	Geotextile used as Sub-layer for the Launching Apron	198
	11.3.4	Transport, Storage, Handling and Placing of Geotextile	198
	11.3.5	Geotextile Testing	199

CHAPTER TWELVE: Rip-Rap and Bonded Stones

12.1	Rip-rap		201
12.2	Bonded Rip-Rap		202
	12.2.1	Cement-Grouted Rip-Rap	202
	12.2.2	Bitumen-Grouted Rip-Rap	205
12.3	Stone Fill Rip-Rap		207

CHAPTER THIRTEEN: Wiremesh Mattressing

13.1	General Description		211
13. 2	Material Specifications		211
	13.2.1	Wiremesh Cages	211
	13.2.2	Brick Fill	211
	13.2.3	Stone Fill	211
13.2.4	Grading Tests at Point of Delivery		214

CHAPTER FOURTEEN: Protection Elements

14.1	General	215
14.2	Boulders/Rocks/Stones	215
14.3	Pre-cast Concrete Blocks	215
14.4	Gabions/ Wire-netting crates	216
14.5	Geotextile Bags	217
14.6	Porcupines	218
14.7	Grout Filled Mattress	218
14.8	Technical Specification for Concrete Blocks	219
	14.8.1 General	219
	14.8.2 Cement	219
	14.8.3 Fine Aggregates	219
	14.8.4 Coarse Aggregates	220
	14.8.5 Water	220
	14.8.6 Nominal Concrete Mix for Blocks	220
	14.8.7 Quality Control	220
	14.8.8 Production	221
	14.8.9 Curing Concrete and Protection	221

CHAPTER FIFTEEN: Bank Revetment

15.1	General Description	223
15.2	Construction Sequence	223
15.3	Preparation of Toe Foundation	223
15.4	Toe Protection	224
15.5	Revetment Material (protection element)	225
15.6	Apron Material/Toe Protection (protection elements)	225
	15.6.1 Toe Protection/Apron Material below DLWL	225
	15.6.2 Revetment above DLWL	225
15.7	Repairs to Bank Revetments	226

CHAPTER SIXTEEN: Groyne

16.1	Impermeable Groyne	229
	16.1.1 Construction of impermeable RCC groyne	229
	16.1.2 Components of RCC groyne	229
	16.1.3 Construction Sequence of RCC groyne	232
16.2	Permeable Groyne	233
	16.2.1 General	233

CHAPTER SEVENTEEN: Pile Installation Procedure and Equipment

17.1	Pile Categories	235
17.2	Categories of Pile Driving/Installation	235
17.3	Pile Installation by Vibration	236
17.4	Pile Installation by Hammer	237
17.5	Installation of Timber Piles	237

17.6 Installation of Pre-Cast Reinforced Concrete Piles 238
17.7 Installation of Steel Piles 240
17.8 Estimation of Piling Hammer Capacity 241
17.9 Pile Installation Work 246
17.9.1 General 246
17.9.2 Tubular Steel Piles 247
17.9.3 Steel Sheet Piling 247
17.9.4 Concrete Piles / Concrete Sheet-piles 248
17.9.5 Pile Butts 248
17.9.6 Surveying and Tolerances 249
17.9.7 Observations During Installation 249
17.9.8 Records of installation 250
17.10 Board Concrete Piles (In-situ Concrete Piles) 251
17.10.1 Definition 251
17.10.2 General 251
17.10.3 Location and Tolerances 252
17.10.4 Materials 252
17.10.5 Boring Work 253
17.11 Obstacles during Drilling 254
17.12 Concreting 255
17.13 Records of Installation 256
17.14 Controls and Acceptances 256

CHAPTER EIGHTEEN: Falling Aprons 257
257
18.1 General Description 257
18.2 Quarried Rock/Stone Boulders 258
18.3 Cement Concrete Blocks 260
18.4 Geotextile Bags 260
18.5 Selection of Construction Methods 260
18.5.1 Construction Methodology 261
18.5.2 Construction Equipment and Materials 261
18.5.3 Methods for Construction 262
18.6 Riverbank Protection by Mass Dumping 267
18.7 Riverbank Protection by Arial Coverage 267
18.8 Quality Control 268
18.8.1 Under Water Diving Activities
18.8.2 Records

CHAPTER NINTEEN: Monitoring and Maintenance

19.1 General 269
19.2 River Monitoring 269
19.3 Operation Monitoring 270
19.4 River Survey 271
19.5 Maintenance 271

REFERENCES 273

APPENDIX – A: **Experience of Selected Bank Protection Works on Major Rivers in Bangladesh**

APPENDIX – B: **Design Examples of Selected Bank Protection Works on Major Rivers in Bangladesh**

GLOSSARY

Abrasion:	Mechanical wearing away by rock material transported by water or wind or Removal of streambank material due to entrained sediment, ice, or debris rubbing against the bank.
Abutment:	A substructure unit composed of stone, concrete, brick, or timber supporting the end of a single span or the extreme end of a multispan superstructure and, in general, retaining or supporting the approach embankment placed in contact therewith.
Acceleration:	Acceleration is the time rate of change in magnitude or direction of velocity vector. Units are meters per second per second, m/s^2 (feet per second per second (ft/s^2)). It is a vector quantity. Acceleration has components both tangential and normal to the streamline, the tangential component embodying the change in magnitude of the velocity, and the normal component reflecting a change in direction.
Adaptive Approach	Adaptive approach means that interventions are planned according to observed and predicted river behavior.
Aggradation:	General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.
Aggrading:	An alluvial river is 'aggrading' when its bed rises because more sediment enters the river than the river can transport.
Alluvial channel:	Channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.
Alluvial fan:	A fan-shaped deposit of material at the place where a stream emerges from a narrow valley of high slope onto a plain or broad valley of low slope. An alluvial cone is made up of the finer materials suspended in flow while a debris cone is a mixture of all sizes and kinds of materials.
Alluvial stream:	A stream which has formed its channel in cohesive or non-cohesive materials that have been and can be transported by the stream.
Alluvium:	Unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan or delta.
Alternating bars:	Elongated deposits found alternately near the right and left banks of a channel.
Altimetry:	Technique to measure altitudes (heights, elevations).
Anabranch:	Individual channel of an anabranched stream.
Anabranching	Formation of a river course with multiple channels, divided by islands, or sometimes bars, that are larger in relation to channel width.
Anabranched stream:	A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; individual islands or bars are wider than about

three times water width; channels are more widely and distinctly separated than in a braided stream.

Anastomosing stream:	An anabranched stream.
Angle of repose:	The angle of slope formed by particulate material under the critical equilibrium condition of incipient sliding or the maximum angle (as measured from the horizontal) at which gravel or sand or granular particles can stand.
Annual flood:	The maximum flow in one year (may be daily or instantaneous)
Apparent Opening Size (AOS):	A measure of the largest effective opening in a filter fabric or geo-textile (sometimes referred to as engineering fabrics), as measured by the size of a glass bead where five percent or less by weight will pass through the fabric (formerly called the equivalent opening size, EOS).
Apron:	Protective material placed on a streambed to resist scour.
Apron, launching:	An apron designed to settle and protect the side slopes of a scour hole after settlement.
Armor (armoring)	Surfacing of channel bed, banks, or embankment slope to resist erosion and scour. (a) Natural process whereby an erosion- resistant layer of relatively large particles is formed on a streambed due to the removal of finer particles by streamflow; (b) placement of a covering to resist erosion.
Armour unit or stone	A relatively large quarry stone or concrete shape that is selected to fit geometric characteristics and density. In normal cases it is used as primary wave protection and is placed in thickness of at least two units.
Articulated concrete	Rigid concrete slabs which can move without separating mattress: as scour occurs; usually hinged together with corrosion- resistant cable fasteners; primarily placed for lower bank protection.
Asphalts:	Black to dark-brown, solid or semi solid materials which, when heated, gradually becomes liquid. They consist mostly of natural bitumen, bitumen obtained from the refining process of crude oil, or a mixture of the two, or a mixture of the two with crude oil and its derivatives.
Astronomical tide:	The tidal levels and character which would result from gravitational effects e.g. of the earth, sun and moon, without any atmospheric influences.
Average velocity	Velocity at a given cross section determined by dividing discharge by cross sectional area.
Avulsion	A sudden change in the channel course that usually occurs when a stream breaks through its banks; usually associated with a flood or a catastrophic event.
Axisymmetrical	Related to idealized bends with uniform curvature.
Backfill:	Material placed adjacent to an abutment, pier, retaining wall or other structure or part of a structure to fill the unoccupied portion of the foundation excavation.

Backwater:	The increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions. A bridge or other structure that obstructs or constricts the free flow of water in a channel induces it.
Backwater area:	The low-lying lands adjacent to a stream that may become flooded due to backwater.
Bank:	The sides of a channel between which the flow is normally confined.
Bank, left, (right)	The side of a channel as viewed in a downstream direction.
Bankful Discharge:	Discharge that, on the average, fills a channel to the point of overflowing.
Bank Protection:	Engineering works for the purpose of protecting streambanks from erosion.
Bank Revetment:	Erosion-resistant materials placed directly on a streambank to protect the bank from erosion or a layered system placed on the filters on a sloping surface as protection against hydraulic forces and scouring.
Bar:	An elongated deposit of alluvium within a channel, not permanently vegetated.
Base Floodplain:	The floodplain associated with the flood with a 100-year recurrence interval.
Bathymetry	Spatial depth distribution, usually defined as the submerged bed topography with respect to a sloping idealized water level surface for a certain specified constant discharge.
Bathymetric Survey:	Mapping of riverbed or seabed elevations.
Batter:	The inclination of a surface in relation to a horizontal or vertical plane or occasionally in relation to an inclined plane. Batter is commonly designated upon bridge detail plans as so many inches (cms) to one foot (meter).
Beach:	The zone of unconsolidated material that extends landward from low water line to the place where there is marked change in material or physiographic form, or to the line of permanent vegetation (usually the effective limit of storm waves). The sea-ward limit of a beach, unless otherwise specified, is the mean low water line. A beach includes foreshore and backshore.
Bed:	The bottom of a channel bounded by banks.
Bed form:	A recognizable relief feature on the bed of a channel, such as a ripple, dune, plane bed, antidune, or bar. Bed forms are a consequence of the interaction between hydraulic forces (boundary shear stress) and the bed sediment.
Bed layer:	A flow layer, several grain diameters thick (usually two) immediately above the bed.
Bed load:	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer (contact

load).

Bed load discharge (or bed load):	The quantity of bed load passing a cross section of a stream in a unit of time.
Bed material:	Material found in and on the bed of a stream (May be transported as bed load or in suspension).
Bed shear (tractive force):	The force per unit area exerted by a fluid flowing past a stationary boundary.
Bed slope:	The inclination of the channel bottom.
Bed topography	Spatial bed level distribution with respect to a horizontal datum.
Bend scour:	Scour in outer bend.
Berm (Berme):	The line, whether straight or curved, which defines the location where the top surface of an approach embankment or causeway is intersected by the surface of the side slope. This term is synonymous with "Roadway Berm". A horizontal bench located at the toe of the slope of an approach cut, embankment or causeway to strengthen and secure its underlying material against sliding or other displacement into an adjacent ditch, borrow pit, or other artificial or natural low lying area.
Bifurcation	Point where a channel or river splits into two channels or rivers.
Bitumens:	Liquid, semi solid or solid hydrocarbon mixtures of natural or pyrogenic origin, or both, completely soluble in carbon disulphide (CS_2).
Bituminous materils:	(asphaltic bitumen); A material consisting mostly of bitumen and containing also other substances, organic and inorganic.
Bituminous emulsion:	A liquid mixture of bitumen droplets suspended in water containing a small proportion of emulsifier (e.g. soap, casein, resin, etc). The emulsion must contain 50% bitumen.
Blanket:	A protection against stream scour placed adjacent to abutments and piers, and covering the streambed for a distance from these structures considered adequate for the stream flow and the streambed conditions or in other words material covering all or a portion of a streambank to prevent erosion.
Boulder:	A rock fragment whose diameter is greater than 250 mm.
Braiding	Formation of a river course with multiple river channels, divided by bars that have a size on the order of the channel width.
Braided stream:	A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; a braided stream has the aspect of a single large channel within which are subordinate channels
Bridge opening:	The cross-sectional area beneath a bridge that is available for conveyance of

	water.
Bridge Waterway:	The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.
Bulk density:	Density of the water sediment mixture (mass per unit volume), including both water and sediment.
Bulkhead:	A vertical, or near vertical, wall that supports a bank or an embankment; also may serve to protect against erosion
Bulking:	Increasing the water discharge to account for high concentrations of sediment in the flow.
Bulle effect	Disproportionate sediment transport into offtake channels due to helical flow.
Catchment:	Same as drainage basin.
Causeway:	Rock or earth embankment carrying a roadway across water.
Caving:	The collapse of a bank caused by undermining due to the action of flowing water.
Cellular block:	Interconnected concrete blocks with regular cavities placed mattress: directly on a streambank or filter to resist erosion. The cavities can permit bank drainage and the growth of vegetation where synthetic filter fabric is not used between the bank and mattress.
Channel:	The bed and banks that confine the surface flow of a stream.
Channelization:	Straightening or deepening of a natural channel by artificial cutoffs, grading, flow-control measures, or diversion of flow into an engineered channel.
Channel diversion:	The removal of flows by natural or artificial means from a natural length of channel.
Channel pattern:	The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, and anabranching.
Channel profile:	Longitudinal section of a channel.
Channel process:	Behavior of a channel with respect to shifting, erosion and sedimentation.
Check dam:	A low dam or weir across a channel used to control stage or degradation.
Chezy coefficient;	Coefficient of hydraulic roughness used for free surface flows.
Choking (flow):	Excessive constriction of flow that may cause severe backwater effect.
Clay (mineral):	A particle whose diameter is in the range of 0.00024 to 0.004 mm.
Clay plug:	A cutoff meander bend filled with fine grained cohesive sediments.

Clear-water scour: Scour at a pier or abutment (or contraction scour) when there is no movement of the bed material upstream of the bridge crossing at the flow causing bridge scour.

Climate: Temperature, precipitation type, seasonal duration and occurrence, mean annual rainfall, rainfall intensity, frequency.

Closed-conduit flow: Flow in a pipe, culvert, etc. where there is a solid boundary on all four sides. Examples are pipes, culverts, and box culverts. Flow conditions in a closed conduit may occur as gravity full flow, full pressure flow, or partly full (open-channel flow).
.

Cobble: A fragment of rock whose diameter is in the range of 64 to 250 mm.

Composite Lining: Combination of lining materials in a given cross section (i.e., riprap low-flow channel and vegetated upper banks).

Concrete revetment: Un-reinforced or reinforced concrete slabs placed on the channel bed or banks to protect it from erosion.

Confluence: The junction of two or more streams.

Confluence scour: Scour occurring where two channels or rivers join.

Constriction: A natural or artificial control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the upstream water surface elevation is related to discharge.

Constriction scour Scour occurring in a narrower channel or river sections.

Contact load: Sediment particles that roll or slide along in almost continuous contact with the streambed (bed load).

Continental shelf: (1)The zone bordering a continent extending from the line of permanent immersion to the depth, usually about 100 m to 200 m, where there is a marked or rather steep descent toward the great depths of the ocean.(2) the area under active littoral processes during the HOLOCENE period. (3) the region of the oceanic bottom that extends outward from the shoreline with an average slope of less than 1:100, to a line where the gradient begins to exceed 1:40 (the CONTINENTAL SLOPE).

Contraction: The effect of channel or bridge constriction on flow streamlines.

Contraction scour: Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scours results from a contraction of the flow area at the bridge, which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.

Conveyance: Geometrical property of a cross-section that determines the relation between the discharge and cross-sectional averaged flow velocity.

Convective The change in velocity (either magnitude or direction or both) with distance.

acceleration:

Coriolis force: The inertial force caused by the Earth's rotation that deflects a moving body to the right in the Northern Hemisphere.

Countermeasure: A measure intended to prevent, delay or reduce the severity of hydraulic problems.

Crib: A frame structure filled with earth or stone ballast, designed to reduce energy and to deflect stream flow away from a bank or embankment.

Critical shear stress: The minimum amount of shear stress required initiating soil particle motion.

Crossing: The relatively short and shallow reach of a stream between bends; also crossover or riffle.

Cross-section: A section normal to the trend of a channel or flow.

Current: Water flowing through a channel.

Current meter: An instrument used to measure flow velocity.

Cut bank: The concave wall of a meandering stream.

Cut-off: A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop (neck cutoff) or across a point bar (chute cutoff).

Cut-off wall: A wall, usually of sheet piling or concrete, that extends down to scour-resistant material or below the expected scour depth.

Cyclone: System of winds that rotates about a centre of low atmospheric pressure. Rotation is clockwise in the Southern Hemisphere and anti-clockwise in the Northern Hemisphere. In the Indian Ocean, the term refers to the powerful storms called HURRICANES in the Atlantic.

Daily discharge: Discharge averaged over one day (24 hours).

Datum: A defined vertical reference level (for water level, bed level, or land elevations).

Debris: Floating or submerged material, such as logs, vegetation, or trash, transported by a stream.

Degradation (bed): A general and progressive (long-term) lowering of the channel bed due to erosion, over a relatively long channel length.

Degrading: An alluvial river is ‘degrading’ when its bed goes down because less sediment enters the river than the river can transport.

Density current: The phenomenon of gravity flow of a liquid relative to another liquid or of relative flow within a liquid medium due to difference in density. Note: the saltwater wedge is a specific case of density current when stratification occurs between identifiable flow masses.

Depth of scour:	The vertical distance a streambed is lowered by scour below a reference elevation.
Design flow (design flood):	The discharge that is selected as the basis for the design or evaluation of a hydraulic structure.
Design discharge:	Discharge at a specific location defined by an appropriate return period to be used for a design purpose.
Dike:	An impermeable linear structure for the control or containment of over bank flow. A dike-trending parallel with a stream bank differs from a levee in that it extends for a much shorter distance along the bank, and it may be surrounded by water during floods.
Dike (groin, spur, jetty):	A structure extending from a bank into a channel that is designed to: (a) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (b) deflect erosive current away from the stream bank (impermeable dike).
Discharge:	Volume of water passing through a channel during a given time. The quantity of water moving past a given plane (cross section) in a given unit of time. Units are cubic meters per second, m^3/s (cubic feet per second, (ft^3/s)). The plane or cross section must be perpendicular to the velocity vector.
Diurnal tide:	These have one high water and one low water on each lunar day. They are common in enclosed bodies of water such as the Gulf of Mexico, the Caribbean Sea, and the waters of the East Indies. Diurnal tides are usually quite small and irregular.
Dolphin:	Cluster of piles driven in water for mooring purposes or for protection against floating objects.
Dominant discharge:	(a)The discharge of water which is of sufficient magnitude and frequency to have a dominating effect in determining the characteristics and size of the stream course, channel, and bed; (b) That discharge which determines the principal dimensions and characteristics of a natural channel. The dominant formative discharges depends on the maximum and mean discharge, duration of flow, and flood frequency. For hydraulic geometry relationships, it is taken to be the bank full discharge that has a return period of approximately 1.5 years in many natural channels.
Drainage basin:	An area confined by drainage divides, often having only one outlet for discharge (catchments, watershed).
Dredging	Removal of any soil by bank-sided or floating equipment below water level irrespective of the method employed.
Drift:	Alternative term for vegetative "debris."
Dune:	Bedform which scales with water depth.
Easting:	East coordinate in the BTM grid, given in m (6 digits) or km (3 digits). Easting = 500 km + the distance (positive towards east) from the 90° east meridian. Hence, as per definition, the 90° east meridian has an easting of

	500. For example, the easting of Gulshan (at the SOB primary benchmark) is 542.
Ebb current (Ebb):	The seaward movement of water along a tidal channel.
Eddy current:	A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.
El Nino:	Warm equatorial water which flows southward along the coast of Peru and Ecuador during February and March of certain years. It is caused by poleward motion of air and unusual water temperature patterns in the Pacific Ocean, which cause coastal downwelling, leading to the reversal in the normal north-flowing cold coastal currents. During many El Nino years, storms, rainfall, and other meteorological phenomenon in the Western Hemisphere are measurably different than during non-El Nino years.
Entrenched stream:	Stream cut into bedrock or consolidated deposits.
Ephemeral stream:	A stream or reach of stream that does not flow for parts of the year. As used here, the term includes intermittent streams with flow less than perennial.
Epoxy:	A synthetic resin which causes or hardens by chemical reaction between components which are mixed together shortly before use.
Ephemeral scour:	Scour depth in sand-bed stream with dune bed about which live bed pier scour level fluctuates due to variability in bed material transport in the approach flow.
Erosion:	Displacement of soil particles due to water or wind action.
Erosion control matting:	Fibrous matting (e.g., jute, paper, etc.) placed or sprayed on a stream- bank for the purpose of resisting erosion or providing temporary stabilization until vegetation is established.
Estuary:	An estuary is a partially enclosed body of water in the lower reaches of a river which is freely connected with the sea and which may generally receive fresh water supplies from upland drainage areas.
Equatorial Currents:	(1)Ocean currents flowing westerly near the equator. There are two such currents in the Atlantic and the Pacific Oceans. The one to the north of the equator is called the North Equatorial Current and the one to the south is called the South Equatorial Current. Between theses two currents there is an easterly flowing stream known as the Equatorial Countercurrent. (2) Tidal currents occurring semimonthly as a result of the moon being over the equator. At these times the tendency of the moon to produce DIURNAL INEQUALITY in the current is at a minimum.
Fabric mattress:	Grout-filled mattress used for stream bank protection.
Factor of safety:	A factor or allowance predicted by common engineering practice upon the failure stress or stresses assumed to exist in a structure or a member or a part thereof. Its purpose is to provide a margin in the strength, rigidity,

deformation and endurance of a structure or its component parts compensating for irregularities existing in structural materials and workmanship, uncertainties involved in mathematical analysis and stress distribution, service deterioration and other unevaluated conditions.

Falling apron: Toe protection of granular material, such as concrete blocks or boulders, placed directly on the existing sub-soil or riverbed (i.e. without filter).

Fall velocity: The velocity at which a sediment particle falls through a column of still water.

Fascia: An outside, covering member designed on the basis of architectural effect rather than strength and rigidity although its function may involve both.

Fascine: A matrix of willow or other natural material woven in bundles and used as a filter. Also, a stream bank protection technique consisting of wire mesh or timber attached to a series of posts, sometimes in double rows; the space between the rows may be filled with rock, brush, or other materials.

Fender: A replaceable device for protecting structure from damage caused by impact from floating bodies.

Fetch: Length of uninterrupted contact between water surface and atmosphere, allowing water waves to grow by transfer of wind energy or the area in which waves are generated by wind having a rather constant direction and speed; sometimes used synonymously with fetch length.

Fetch length: The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

Fill slope: Side or end slope of an earth-fill embankment. Where a fill-slope forms the stream-ward face of a spill-through abutment, it is regarded as part of the abutment.

Filter: Layer of fabric (geo-textile) or granular material (sand, gravel, or graded rock) placed between bank revetment (or bed protection) and soil for the following purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; (2) to prevent the revetment from sinking into the soil; and (3) to permit natural seepage from the stream bank, thus preventing the buildup of excessive hydrostatic pressure.

Filter blanket: A layer of graded sand and gravel laid between fine-grained material and riprap to serve as a filter.

Filter fabric: Geosynthetic fabric that serves the same purpose as a granular filter blanket.

Fine sediment load: That part of the total sediment load that is composed of particle sizes finer than those represented in the bed (wash load). Normally, the fine-sediment load is finer than 0.062 mm for sand-bed channels. Silt, clay and sand could be considered wash load in coarse gravel and cobble-bed channels.

Flanking: Erosion around the landward end of a stream stabilization countermeasure.

Flashy stream: Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Typically associated with mountain streams or highly disturbed urbanized catchments. Most flashy streams are ephemeral, but some are perennial.

Flexible Lining: A channel lining material having the capacity to adjust to settlement; typically constructed of a porous material that allows infiltration and exfiltration.

Float tracking Measurement of surface flow velocities and flow lines by recording subsequent positions of floating objects.

Flood-frequency curve: A graph indicating the probability that the annual flood discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude.

Floodplain: A nearly flat, alluvial low land bordering a stream, that is subject to frequent inundation by floods.

Flow hazard: Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.

Flow slide: Saturated soil materials that behave more like a liquid than a solid. A flow slide on a channel bank can result in a bank failure or bank erosion by means of relatively fast mass failure (liquefaction), resulting in deep bays in the bank line with a narrow neck.

Flush causeway: A causeway at bed level of a stream.

Fluvial geomorphology: The science dealing with the morphology (form) and dynamics of streams and rivers.

Fluvial system: The natural river system consisting of (1) the drainage basin, watershed, or sediment source area, (2) tributary and mainstem river channels or sediment transfer zone, and (3) alluvial fans, valley fills and deltas, or the sediment deposition zone.

Free board: The vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.

Froude number: A dimensionless number that represents the ratio of inertial to gravitational forces in open channel flow. The Froude Number is also the ratio of the flow velocity V to the celerity gL C = of a small gravity wave in the flow.

Gabion: A basket or compartmented rectangular container made of wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable unit with which flow- and erosion-control structures can be built.

Gauging: Measurement at a fixed point.

General scour: General scour is a lowering of the streambed across the stream or waterway at the bridge. This lowering may be uniform across the bed or non-uniform.

	That is, the depth of scour may be deeper in some parts of the cross section. General scour may result from contraction of the flow or other general scour conditions such as flow around a bend.
Geodetic Survey:	Mapping of geodetic coordinates (of bench marks, reference stations, etc).
Geomorphology/ morphology:	The science that deals with the form of the Earth, the general configuration of its surface, and the changes that take place due to erosion and deposition.
Global Positioning System: (GPS)	A navigational and positioning system developed by the U.S.Department of Defense, by which the location of a position on or above the Earth can be determined by a special receiver at that point interpreting signals received simultaneously from several of a constellation of special satellites.
Grade-control structure (sill, check dam):	Structure placed bank to bank across a stream channel usually (with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or head-cutting.
Graded stream:	A geomorphic term used for streams that have apparently achieved a state of equilibrium between the rate of sediment transport and the rate of sediment supply throughout long reaches.
Gravel:	A rock fragment whose diameter ranges from 2 to 64 mm.
Grillage:	A plat-form like construction or assemblage used to insure distribution of loads upon unconsolidated soil material.
Groin/Groyne:	A structure built from the bank of a stream in a direction transverse to the current to redirect the flow or reduce flow velocity. Many names are given to this structure, the most common being "spur," "spur dike," "transverse dike," "jetty," etc. Groins may be permeable, semi-permeable, or impermeable.
Grout:	A fluid mixture of cement and water or of cement, sand, and water used to fill joints and voids.
Guide bank:	A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guidebanks extend downstream from the bridge (also spur dike).
Hard point:	A streambank protection structure whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank. Hard points also occur naturally along streambanks as passing currents remove erodible materials leaving non-erodible materials exposed.
Head cutting:	Channel degradation associated with abrupt changes in the bed elevation (headcut) that generally migrates in an upstream direction.
Helical flow:	Three-dimensional movement of water particles along a spiral path in the general direction of flow. These secondary-type currents are of most significance as flow passes through a bend; their net effect is to remove soil particles from the cut bank and deposit this material on a point bar.
High water	The highest water level reached by the water surface at a place in course of

(H.W): tidal — one tidal oscillation.

High water slack: The temporary state of rest for the water mass at a place as the tidal current changes over from flood to ebb.

Hurricane: An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 33.5 m/sec (75 mph or 65 knots) for several minutes or longer at some points. TROPICA STORM is the term applied if maximum winds are less than 33.5 m/sec but greater than a whole gale (63 mph or 55 knots). Term is used in the Atlantic, Gulf of Mexico, and eastern Pacific.

Hydraulics: The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures, and the ground.

Hydraulic loads Forces due to action of water (hydrostatic or hydrodynamic).

Hydraulic Problem: An effect of streamflow, tidal flow, or wave action such that the integrity of the highway facility is destroyed, damaged, or endangered.

Hydraulic radius: The cross-sectional area of a stream divided by its wetted perimeter. The hydraulic radius is a length term used in many of the hydraulic equations that is determined by dividing the flow area by the length of the cross section in contact with the water (wetted perimeter). The hydraulic radius is in many of the equations to help take into account the effects of the shape of the cross section on the flow. For example, the hydraulic radius for a circular pipe flowing full is equal to the diameter of the pipe divided by four (D/4).

Hydraulic structures: The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

Hydrograph: The graph of stage or discharge against time.

Hydrology: The science concerned with the occurrence, distribution, and circulation of water on the earth.

Imbricated: In reference to stream bed sediment particles, having an overlapping or shingled pattern.

Impinging flow: Flow attacking a river bank.

Icing: Masses or sheets of ice formed on the frozen surface of a river or floodplain. When shoals in the river are frozen to the bottom or otherwise dammed, water under hydrostatic pressure is forced to the surface where it freezes.

Incised reach: A stretch of stream with an incised channel that only rarely overflows its banks.

Incised stream: A stream which has deepened its channel through the bed of the valley floor, so that the floodplain is a terrace.

Incipient motion: The condition that exists just prior to the movement of a particle within a flow field. Under this condition, any increase in any of the factors responsible for particle movement will cause motion.

Invert:	The lowest point in the channel cross-section or at flow control devices such as weirs, culverts, or dams.
Island:	A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Islands originate by establishment of vegetation on a bar, by channel avulsion, or at the junction of minor tributary with a larger stream.
Jack:	A device for flow control and protection of banks against lateral erosion consisting of three mutually perpendicular arms rigidly fixed at the center. Kellner jacks are made of steel struts strung with wire, and concrete jacks are made of reinforced concrete beams.
Jack field:	Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.
Jetty:	(a) An obstruction built of piles, rock, or other material extending from a bank into a stream, so placed as to induce bank building, or to protect against erosion; (b) A similar obstruction to influence stream, lake, or tidal currents, or to protect a harbor (also spur).
Knot:	The unit of speed used in navigation equal to one nautical mile (6,076.115 ft. or 1,852 m) per hour.
Laminar flow:	In laminar flow, the mixing of the fluid and momentum transfer is by molecular activity.
Lateral erosion:	Erosion in which the removal of material is extended horizontally as contrasted with degradation and scour in a vertical direction.
Launching:	Release of undercut material (stone riprap, rubble, slag, etc.) downslope or into a scoured area.
Launching apron:	Integrated and articulated toe protection, i.e. mattress systems, such as sand-filled geo-textile bags, concrete-filled geo-textile bags or concrete blocks linked to a strong geo-textile fabric, placed on prepared slopes and a filter layer above and below water or in a horizontal excavation above DLW (Design Low Water).
Levee:	An embankment, generally landward of top bank, that confines flow during high-water periods, thus preventing overflow into lowlands.
Leeward:	The direction towards which the wind is blowing; the direction towards which waves are traveling.
Live-bed scour:	Scour at a pier or abutment (or contraction scour) when the bed material in the channel upstream of the bridge is moving at the flow causing bridge scour.
Local acceleration:	Local acceleration is the change in velocity (either in magnitude and direction or both) with time at a given point or cross section.
Local scour:	Removal of material from around piers, abutments, spurs, and embankments

	caused by an acceleration of flow and resulting vortices induced by obstructions to the flow or scour downstream from a local structure or obstruction.
Load (sediment load):	Amount of sediment being moved by a stream.
Lower bank:	That portion of a streambank having an elevation less than the mean water level of the stream.
Low control structure:	A structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.
Low water (L.W): (tidal)	The lowest level reached by the water surface at a place in course of one tidal oscillation.
Low water slack: (tidal)	The temporary state of rest for the water mass at a place as the tidal current changes over from ebb to flood.
Mass failure:	Bank erosion process in which large portions of the bank collapse into the river during short events.
Mathematical model:	A numerical representation of a flow situation using mathematical equations (also computer model).
Mattress:	A blanket or revetment of materials interwoven or otherwise lashed together and placed to cover an area subject to scour. .
Meander:	One curved portion of a sinuous or a winding stream channel, consisting of two consecutive loops, flowing clockwise and the other counter-clockwise.
Meander amplitude:	The distance between points of maximum curvature of successive meanders of opposite phase in a direction normal to the general course of the meander belt, measured between center lines of channels.
Mean High Water Springs (MHWS):	The average height of the high water occurring at the time of spring tide.
Mean High Water (MHW):	The average height of the high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.
Mean Higher High Water (MHHW):	The average height of the higher high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value.
Mean Low Water (MLW):	The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the lower low water heights are included in the

average where the tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

Mean Lower Low Water (MLLW):	The average height of the lower low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. Frequently abbreviated to LOWER LOW WATER.
Mean Range of Tide:	The difference in height between MEAN HIGH WATER and MEAN LOW WATER.
Mean sea level (M.S.L):	The average height of the surface of the sea in all states of oscillation; this is taken equivalent to the level which would have existed in the absence of tidal forces. (2) The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. Not necessarily equal to MEAN TIDE LEVEL.
Mean tide level: (M.T.L)	It is the level halfway between the high water level and low water level of a tide.
Meander belt:	The distance between lines drawn tangent to the extreme limits of successive fully developed meanders.
Meandering stream:	A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops. The channel generally exhibits a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.
Median diameter:	The particle diameter of the 50th percentile point on a size distribution curve such that half of the particles (by weight, number, or volume) are larger and half are smaller (D_{50}.).
Mid channel bar:	A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.
Middle bank:	The portion of a streambank having an elevation approximately the same as that of the mean water level of the stream.
Migration:	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
Mixed tide:	Tide characterized by two markedly unequal high waters, or two markedly unequal low waters, or both on each lunar day, during most of the month.
Morphodynamics	Study of the time-dependent changes in the forms of alluvial beds and their underlying processes. The term is also used as a synonym for morphological behavior.
Morphology	Branch of geomorphology, studying the forms of alluvial beds of water bodies and their ongoing changes by erosion and sedimentation. The term is also used as a synonym for the bed topography or the shape of a river.
Mud:	A soft, saturated mixture mainly of silt and clay.

Natural levee:	A low ridge that slopes gently away from the channel banks that is formed along streambanks during floods by deposition.
Neap tide:	Tide of decreased range occurring semimonthly as the result of the moon being in quadrature. The NEAP RANGE of the tide is the average semidiurnal range occurring at the time of neap tides and most conveniently computed from the harmonic constants. The NEAP RANGE is typically 10 to 30 percent smaller than the mean range where the type of tide is either semidiurnal or mixed and is of no practical significance where the type of tide is DIURNAL. The average height of the high waters of the neap tide is called NEAP HIGH WATER or HIGH WATER NEAPS (MHWN), and the average height of the corresponding LOW WATER is called NEAP LOW WATER or LOW WATER NEAPS (MLWN).
Nominal diameter:	Equivalent spherical diameter of a hypothetical sphere of the same volume as a given sediment particle.
Non-alluvial channel:	A channel whose boundary is in bedrock or non-erodible material.
Non-uniform flow:	In non-uniform flow, the velocity of flow changes in magnitude or direction or both with distance. The convective acceleration components are different from zero. Changes occurring over long distances are classified as gradually varied flow. Changes occurring over short distances are classified as rapidly varied flow. Examples are flow around a bend or flow in expansions or contractions.
Normal depth:	The depth of a uniform channel flow.
Normal stage:	The water stage prevailing during the greater part of the year.
Northing:	North coordinate in the BTM grid, given in m (6 digits) or km (3 digits). The northing = the distance to equator minus 2000 km. For example, the northing of gulshan (at the SOB primary bench mark) is 631.
One dimensional flow:	A method of analysis where changes in the flow variables (velocity, depth, etc.) occur primarily in the longitudinal direction. Changes of flow variables in the other two dimensions are small and are neglected.
Open-channel flow:	Open-channel flow is flow with a free surface. Closed-conduit flow or flow in culverts is open-channel flow if they are not flowing full and there is a free surface.
Overbank flow:	Water movement that overtops the bank either due to stream stage or to overland surface water runoff.
Oxbow:	The abandoned former meander loop that remains after a stream cuts a new, shorter channel across the narrow neck of a meander. Often bow-shaped or horseshoe-shaped.
Pavement:	Stream-bank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on stream-banks are concrete, compacted asphalt, and soil-cement.
Paving:	Covering of stones on a channel bed or bank (used with reference to natural

covering).

Peaked stone dike:	Riprap placed parallel to the toe of a stream-bank (at the natural angle of repose of the stone) to prevent erosion of the toe and induce sediment deposition behind the dike.
Perennial stream:	A stream or reach of a stream that flows continuously for all or most of the year.
Permeability:	The property of a material or a substance that describes the degree to which the material is penetrable by liquids or gases. Also the measure of this property.
Permittivity:	The permeability of a geo-textile normal to the fabric per unit thickness of the fabric.
Pheratic line:	The upper boundary of the seepage water surface landward of a stream-bank.
Pile:	An elongated member, usually made of timber, concrete, or steel, that serves as a structural component of a river-training structure.
Pile dike:	A type of permeable structure for the protection of banks against caving; consists of a cluster of piles driven into the stream, braced and lashed together.
Piping:	Removal of soil material through subsurface flow of seepage water that develops channels or "pipes" within the soil bank.
Pixel:	Picture element.
Planform:	Shape on map of banklines or water lines (top view).
Point bar:	An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.
Poised stream:	A stream which, as a whole, maintains its slope, depths, and channel dimensions without any noticeable raising or lowering of its bed (stable stream). Such condition may be temporary from a geological point of view, but for practical engineering purposes, the stream may be considered stable.
Pressure flow:	Flow in a closed conduit or culvert that is flowing full with water in contact with the total enclosed boundary and under pressure.
Probable maximum flood:	A very rare flood discharge value computed by hydrometeorological methods, usually in connection with major hydraulic structures.
Protrusion scour:	Scour immediately upstream from a local structure or obstruction due to local acceleration of the flow.
PWD:	Public Works Department datum. A horizontal datum applied by PWD, BWDB and others. It is defined by a network of SOB and BWDB benchmarks with a specified elevation above PWD. Its zero level is located 0.46 m below the Mean Sea Level (MSL) defined in 1909.

Quarry-run stone:	Stone as received from a quarry without regard to gradation requirements.
Rail bank protection:	A type of countermeasure composed of rock-filled wire fabric supported by steel rails or posts driven into streambed.
Rapid drawdown:	Lowering the water against a bank more quickly than the bank can drain without becoming unstable.
Reach:	A segment of stream length that is arbitrarily bounded for purposes of study.
Recurrence interval:	The reciprocal of the annual probability of exceedence of a hydrologic event (also return period, exceedence interval).
Regime:	The condition of a stream or its channel with regard to stability. A stream is in regime if its channel has reached an equilibrium form as a result of its flow characteristics. Also, the general pattern of variation around a mean condition, as in flow regime, tidal regime, channel regime, sediment regime, etc. (used also to mean a set of physical characteristics of a river).
Regime change:	A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads, or slope.
Regime channel:	Alluvial channel that has attained, more or less, a state of equilibrium with respect to erosion and deposition.
Regime formula:	A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.
Reinforced earth bulkhead:	A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a stream-bank.
Reinforced revetment:	A stream-bank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank.
Relief Bridge:	An opening in an embankment on a floodplain to permit passage of overbank flow.
Remote sensing	Observation or measurement without direct contact. In its broadest sense, the term includes hearing and seeing. In its more restricted operational meaning, it refers to gathering information from great distances and over broad areas, usually through instruments mounted on aircraft or orbiting space vehicle.
Resolution:	Size of smallest detectable detail.
Retard (Retarder structure):	A permeable or impermeable linear structure in a channel parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank. .
Retarded scour	Scour occurring during the fall of the flood.
Retrogressive erosion:	Bank erosion by means of relatively slow mass failure (shear failures), resulting in a crescent-shaped line of bank retreat.
Return period:	Recurrence time, average time interval between subsequent events in which

the conditions are exceeded. When designing a structure, the design 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:	Rigid or flexible armor placed to inhibit scour and lateral erosion. (See bank revetment).
Revetment toe:	The lower terminus of a revetment blanket; the base or foundation of a revetment.
Riffle:	A natural, shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.
Rigid lining:	A lining material with no capacity to adjust to settlement; these lining materials are usually constructed of non-porous material.
Riparian:	Pertaining to anything connected with or adjacent to the banks of a stream (corridor, vegetation, zone, etc.).
Ripple	Bedform which scales with viscous sublayer.
Riprap:	Layer or facing of rock or broken concrete that is dumped or placed to protect a structure or embankment from erosion; also the rock or broken concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.
Riverbank erosion management system:	A comprehensive system that is designed to provide stable living conditions along the banks, protecting existing infrastructure, enhancing economic activities and reducing poverty. The system may initially consist of (i) structural riverbank protection system, (ii) non-structural measures mitigating negative impacts of construction (resettlement, environment), (iii) disaster preparedness and management, (iv) social support to vulnerable poor, and (v) socially and environmentally acceptable joint flood plain management through stakeholders' participation and government agency.
River morphology:	Branch of geomorphology, studying the forms of river bed and their ongoing changes by erosion and sedimentation.
River training:	Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel. Also, any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect current, induce sediment deposition, control scour, or in some other way alter the flow and sediment regimes of the stream.
Rock-and-wire mattress:	A flat wire cage or basket filled with stone or other suitable material and placed as protection against erosion.
Rock window:	An erosion control technique that consists of burying or piling a sufficient supply of erosion-resistant material below or on the existing land surface along the bank, then permitting the area between natural riverbank and the rock to erode until the erosion reaches and undercuts the supply of rocks.

Rotary current:	In the open sea the direction of the tidal current normally veers round the compass and the current does not pass through intervals of slack water. This is called rotary current to distinguish it from the reversing current in a tidal channel.
Rubble:	Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.
Runoff:	Part of precipitation that appears in surface streams of either perennial or intermittent form.
Sack Revetment:	Sacks (e.g., burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material used as protection against erosion.
Salt water wedge:	The wedge like intrusion of a large mass of salt water flowing in from the sea of small tidal range under the fresh water in a tidal waterway where mixing by turbulence is inappreciable.
Saltation load:	Sediment bounced along the streambed by energy and turbulence of flow, and by other moving particles.
Sand:	A rock fragment whose diameter is in the range of 0.062 to 2.0 mm.
Scour:	An erosion of a river, stream, tidal inlet, lake or other water bed area by a current, wash or other water in motion, producing a deepening of the overlying water, or a widening of the lateral dimension of the flow area. Erosion of streambed or bank material due to flowing water; often considered as being localized (see local scour, contraction scour, total scour).
Sediment or Fluvial sediment:	Fragmental material transported, suspended, or deposited by water.
Sediment concentration:	Weight or volume of sediment relative to the quantity of transporting (or suspending) fluid.
Sediment discharge:	The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.
Sediment load:	Amount of sediment being moved by a stream.
Sediment Yield:	The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.
Seepage:	The slow movement of water through small cracks and pores of the bank material.
Seiche:	Long-period oscillation of a lake or similar body of water.

Semi-diurnal tides:	These tides have two nearly equal high waters and low waters respectively in each lunar day or 24 hour 50 minutes. They are found along both coasts of North and South Atlantic Oceans and the Indian Ocean.
Shear stress:	The force developed on the wetted area of the channel that acts in the direction of flow, usually measured as a force per unit wetted area.
Shields parameter:	Parameter expressing the mobility of particles considering the grain size and the prevalent flow conditions.
Shoal:	A relatively shallow submerged bank or bar in a body of water.
Shoaling:	Effect of shallow water on waves.
Significant Wave:	A statistical term relating to the one-third highest waves of a given wave group and defined by their heights and periods. The comparison of the higher waves depends upon the extent to which the lower waves are considered. Experience indicates that a careful observer who attempts to establish the character of the higher waves will record values which approximately fit the definition of the significant wave.
Significant Wave Height:	The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, the average height of the highest one-third of a selected number of waves, this number being determined by dividing the time of record by the significant period. Also CHARACTERISTIC WAVE HEIGHT.
Significant Wave Period:	An arbitrary period generally taken as the period of the one-third highest waves within a wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, this is determined as the average period of the most frequently recurring of the larger well-defined waves in the record under study.
Sill:	(a) A structure built under water, across the deep pools of a stream with the aim of changing the depth of the stream; (b) A low structure built across an effluent stream, diversion channel or outlet to reduce flow or prevent flow until the main stream stage reaches the crest of the structure.
Silt:	A particle whose diameter is in the range of 0.004 to 0.062 mm.
Sinuosity:	The ratio between the thalweg length and the valley length of a stream.
Slack Water: (Slack tide)	The state of a tidal current when its velocity is near zero, especially the moment when a reversing current changes direction and its velocity is zero. Sometimes considered the intermediate period between ebb and flood currents during which the velocity of the current is less than 0.5 m/sec (0.1 knot).
Slide:	Collapse of large portions of river bank typically induced by excessive pore pressure and top loads (mass failure).
Slope (of channel or stream):	Fall per unit length along the channel centerline or thalweg.

Slope protection:	Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.
Sloughing:	Sliding or collapse of overlying material; same ultimate effect as caving, but usually occurs when a bank or an underlying stratum is saturated.
Slope-area method:	A method of estimating unmeasured flood discharges in a uniform channel reach using observed high-water levels.
Slump:	A sudden slip or collapse of a bank, generally in the vertical direction and confined to a short distance, probably due to the substratum being washed out or having become unable to bear the weight above it.
SLW:	Standard Low Water datum: A datum representing the water level that is exceeded in 95% of the time. The slope of the datum (approximately) follows the dry season slope of the river surface, inclining downwards towards the sea. The present SLW in Bangladesh was defined by a number of gauge locations by BIWTA in 1990.
Soil cement:	A designed mixture of soil and Portland cement compacted at a proper water content to form a blanket or structure that can resist erosion.
Soil migration:	The transportation of finer soil particles within a soil mass.
Soil piping:	The process by which soil particles are washed in or through pore spaces in filters.
Sorting:	Progressive reduction of size (or weight) of particles of the sediment load carried down a stream.
Spill-through abutment:	A bridge abutment having a fill slope on the streamward side. The term originally referred to the "spill-through" of fill at an open abutment but is now applied to any abutment having such a slope.
Spread footing:	A pier or abutment footing that transfers load directly to the earth.
Spur:	A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.
Spur dike:	Same as guide bank
Stability:	A condition of a channel when, though it may change slightly at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years.
Stable channel:	A condition that exists when a stream has a bed slope and cross section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion (a graded stream).
Stage:	Water-surface elevation of a stream with respect to a reference elevation.

Standing waves:	Curved symmetrically shaped waves on the water surface and on the channel bottom that are virtually stationary.
Steady flow:	In steady flow, the velocity at a point or cross section does not change with time. The local acceleration is zero.
Stone riprap:	Natural cobbles, boulders, or rock dumped or placed as protection against erosion.
Streamline:	An imaginary line within the flow which is everywhere tangent to the velocity vector.
Stream:	A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.
Streambank erosion:	Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to bank erosion.
Streambank failure:	Sudden collapse of a bank due to an unstable condition such as removal of material at the toe of the bank by scour.
Streambank protection:	Any technique used to prevent erosion or failure of a stream-bank.
Storm surge:	Coastal flooding phenomenon resulting from wind and barometric changes. The storm surge is measured by subtracting the astronomical tide elevation from the total flood elevation (Hurricane surge).
Storm tide:	Coastal flooding resulting from combination from storm surge and astronomical tide (often referred to as storm surge).
Suspended load:	Sediment that is supported by the upward components of the turbulent current in a stream and that stays for an appreciable length of time.
Suspended sediment discharge:	The quantity of sediment passing through a stream cross section above the bed layer in a unit of time suspended by the turbulence of flow (suspended load).
Sub-bed material:	Material underlying that portion of the streambed which is subject to direct action of the flow. Also, substrate.
Sub-critical flow:	Open-channel flow's response to changes in channel geometry depends upon the depth and velocity of the flow. Sub-critical flow (or tranquil flow) occurs on mild slopes where the flow is deep with a low velocity and has a Froude Number less than 1. In sub-critical flow, the boundary condition (control section) is always at the downstream end of the flow reach.
Supercritical flow:	Supercritical flow occurs on steep slopes where the flow is shallow with a high velocity and has a Froude Number greater than 1. In supercritical flow, the boundary condition (control section) is always at the upstream end of the flow reach.

Superelevation: Local increases in water surface on the outside of a bend.

Tail water: Water ponded below the outlet of a culvert, pile, or bridge waterway, thereby reducing the amount of flow through the waterway. Tailwater is expressed in terms of depth.

Tar: A bituminous material obtained in the process of destructive refining of vegetative organic materials such as coal, wood, turf or lignite.

Tetrahedron: Component of river-training works made of six steel or concrete struts fabricated in the shape of a pyramid.

Tetrapod: Bank protection component of pre-cast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5_ with the other three.

Thalweg: The line extending down a channel that follows the lowest elevation of the bed.

Three-dimensional flow: The flow variables can change in all three dimensions, along, across, and in the vertical.

Threshold Value (of a survey equipment): The smallest deviation from zero (of the true value) that can be registered.

Tide: The periodic rise and fall of water in the ocean due to principally gravitational attraction of the sun and the moon on the rotating earth.

Tidal amplitude: Difference between the high water (H.W) or low water (L.W) and the mean sea level (M.S.L) at a given point or the mean elevation of the tide. Generally, half of tidal range.

Tidal channel: The channel (or waterway) in which the flow is subjected to tidal action. Water-way includes one or more tidal channels together with the shallows and the banks at the sides by which the tide at high water is bounded.

Tidal currents: The horizontal forward and backward movement of water in estuaries and bays along the coast as a result of tidal action.

Tidal cycle: One complete rise and fall of the tide.

Tidal day: Time of rotation of the earth with respect to the moon. Assumed to equal approximately 24.84 solar hours in length.

Tidal inlet: A channel connecting a bay or estuary to the ocean.

Tidal passage: A tidal channel connected with the ocean at both ends.

Tidal period: Duration of one complete tidal cycle. When the tidal period equals the tidal day (24.84 hours), the tide exhibits diurnal behavior. Should two complete tidal periods occur during the tidal day, the tide exhibits semi-diurnal behavior.

Tidal prism:	Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.
Tidal range:	Vertical distance between specified low and high tide levels.
Tidal scour:	Scour at bridges over tidal waterways, i.e., in the coastal zone.
Tidal waterways	A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands, inlets or bays, crossings between bays, tidally affected streams, etc.
Tides, astronomical:	Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating earth. Also, daily tides.
Tieback:	Structure placed between revetment and bank to prevent flanking.
Timber or brush mattress:	A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream and weighted with ballast.
Toe of Bank	That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.
Toe protection:	Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.
Topographic Survey:	Mapping of land elevations.
Top soil:	Top layer of soil containing a higher proportion of organic material.
Total scour:	The sum of long-term degradation, general (contraction) scour, and local scour.
Total sediment load:	The sum of suspended load and bed load or the sum of bed material load and wash load of a stream (total load).
Tractive force:	Force developed at the channel bed as a result of the resistance to flow created by the channel section. This force acts in the direction of flow, and is equal to the shear stress on the channel section multiplied by the wetted perimeter. In other words the drag or shear on a streambed or bank caused by passing water which tends to move soil particles along with the stream-flow.
Trench-fill revetment:	Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding stream-bank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion.
Turbulence:	Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.
Turbulent flow:	In turbulent flow the mixing of the fluid and momentum transfer is related to random velocity fluctuations. The flow is laminar or turbulent depending on

the value of the Reynolds Number ($Re = \rho VL/\mu$), which is a dimensionless ratio of the inertial forces to the viscous forces. Here ρ and μ are the density and dynamic viscosity of the fluid, V is the fluid velocity, and L is a characteristic dimension, usually the depth (or the hydraulic radius) in open-channel flow. In laminar flow, viscous forces are dominant and Re is relatively small. In turbulent flow, Re is large; that is, inertial forces are very much greater than viscous forces. Turbulent flows are predominant in nature. Laminar flow occurs very infrequently in open-channel flow.

Two-dimensional flow: A method of analysis where the accelerations can occur in two directions (along and across the flow).

Ultimate scour: The maximum depth of scour attained for a given flow condition. May require multiple flow events and in cemented or cohesive soils may be achieved over a long time period.

Uniform flow: Flow of constant cross section and velocity through a reach of channel at a given time. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow. In uniform flow the velocity of the flow does not change with distance. The convective acceleration is zero. Examples are flow in a straight pipe of uniform cross section flowing full or flow in a straight open channel with constant slope and all cross sections of identical form, roughness and area, resulting in a constant mean velocity. Uniform flow conditions are rarely attained in open channels, but the error in assuming uniform flow in a channel of fairly constant slope and cross section is small in comparison to the error in determining the design discharge.

Unit discharge: Discharge per unit width (may be average over a cross section, or local at a point).

Unit shear force shear stress): The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually in units of stress, Pa (N/m^2) or (lb/ft^2).

Unsteady flow: In unsteady flow, the velocity at a point or cross section varies with time. The local acceleration is not zero. A flood hydrograph where the discharge in a stream changes with time is an example of unsteady flow. Unsteady flow is difficult to analyze unless the time changes are small.

Upper bank: The portion of a stream-bank having an elevation greater than the average water level of the stream.

Upwelling Upwelling is the consistent but small vertical flow of water reflected at the surface as a smooth area on the water surface. Upwelling is different than the boils studied and described by Coleman (1969). The boils surface and last on the average about 10 seconds, then disappear. Boils are the reflection of unsteady flow in the lee of dunes and sand waves. Upwelling is continuous and is the reflection of two streams of water meeting obliquely, one or both slightly changing direction and one accelerating the other if they meet with different speeds. It is a reflection of the pressure in excess of hydrostatic that is developed in the flow in close proximity to the bank. In steady flow for a long period, upwelling is centered over the deepest scour.

Velocity: The time rate of flow usually expressed in m/s (ft/sec). The average velocity

	is the velocity at a given cross section determined by dividing discharge by cross-sectional area. The time rate of movement of a water particle from one point to another. Velocity is a vector quantity, that is it has magnitude and direction.
Vertical alignment:	An abutment, usually with wing walls, that has no fill slope on its stream-ward side.
Vortex:	Turbulent eddy in the flow generally caused by an obstruction such as a bridge pier or abutment (e.g., horseshoe vortex).
Wandering Channel:	A channel exhibiting a more or less non-systematic process of channel shifting, erosion and deposition, with no definite meanders or braided pattern.
Wandering thalweg:	A thalweg whose position in the channel shifts during floods and typically serves as an inset channel that conveys all or most of the stream flow at normal or lower stages.
Wash load:	Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.
Water shed:	Same as drainage basin.
Waterway opening width (area):	Width (area) of bridge opening at (below) a specified stage, measured normal to the principal direction of flow.
Wave downrun:	The down slope flow of water experienced immediately following a wave run-up as the water flows back to the normal water elevation.
Weep hole:	A hole in an impermeable wall or revetment to relieve the neutral stress or pore pressure in the soil.
Windrow revetment:	A row of stone placed landward of the top of an eroding stream-bank. As the windrow is undercut, the stone is launched downslope, thus armoring the bank.
Wire mesh:	Wire woven to form a mesh; where used as an integral part of a countermeasure, openings are of suitable size and shape to enclose rock or broken concrete or to function on fence-like spurs and retards.

LIST OF SYMBOLS

A_{ch}	cross-sectional channel area, m^2
B_{ch}	channel width between bank lines of bankfull discharge at FPL, m
B_{ch}	channel width at upstream of the structure, m
B	protrusion length, m
B	co-efficient of flow condition
b	Wave structure interaction coefficient, dependent on roughness and porosity of protective material
C	cohesion of soil
C	Wave velocity, m/s
C	Chezy coefficient = 18*log 12(h/k_s), $m^{0.5}/s$
C_c	Coefficient of curvature
C_{FA}	flow attack co-efficient
C_s	Shape coefficient
Cu	Coefficient of uniformity
C_v	Coefficient of vertical velocity distribution
c_1	safety factor for extreme channel discharge
c_b	braiding index
D_s	scour depth at design discharge, m
D_n	nominal thickness of protection unit, m
D_n	dimension of cube, m
D	diameter of stone, m
D	deepest known scour, m
d_{50}	grain diameter, mm
d_m	weighted mean diameter of sediment particle, mm
F	fetch length, m
F	Free board, m
Fw	porous flow force, kN
Fw	flow force along the slope, kN
Fw	Flow force perpendicular to the slope, kN
F.S.	factor of safety,
F_e	effective fetch length
Fr	Froude number
f	Lacey's silt factor
f	a coefficient related to the amplitude of the wave and slope angle
G	weight of soil mass, kg
g	acceleration due to gravity, m/sec^2
H_{ch}	average water depth within the channel at DWL , m
$\widetilde{H}_s$	wave heights, m
H_s	significant wave height, m
h	water depth, m
h_1	water depth upstream from scour hole, m
h_o	total water depth at thalweg below DLWL, m
h/d	Depth factor
h'_{ch}	average water depth within the channel related to FPL, m
I	slope of energy level
I	hydraulic gradient in the soil

Ip	plasticity index
K	empirical coefficient, $m^{-1/3}s^{2/3}$
K_1	Side slope correction factor
K_1	groyne head
K_2	alignment curvature
K_3	function groynes
K_τ	turbulence factor
K_h	depth factor
K_s	slope factor
k	a coefficient varying from 3.2 for smooth quarry stone to 10 for tetrapods
k_s	roughness parameter, m
k_s	permeability of the subsoil, m/s
k_r	mean equivalent roughness height, m
k'	slope reduction factor
k_g	permeability of the geotextile, m/s
k_s	permeability of the soil, m/s
L	wave length, m
L	length of arc intersecting the bank slope, m
l	arm of force, m
M_B	meander Belt, m
M_L	meander length, m
Ns	sum of normal forces for all the slices,
Q	discharge (m^3/s)
Q_{ch}	design discharge (m^3/s)
Q_b	bankfull river discharge ((m^3/s)
R	hydraulic radius or water depth, m
R	wave runup, m
R	rise of flood above apron, m
r	radius, m
r_c	bend curvature, m
S	Thickness of soling, m
S_s	Specific gravity of stone
T	return period, yr
T	thickness of stone riprap, m
T1	Thickness of stone on prospective slope below bottom of apron, m
T2	Thickness of covering, m
T_s	sum of tangential forces tending to produce movement of the soil along the slip surface circumference, kN
$\widetilde{T}_p$	wave period, s
T_p	peak wave period, s
T_d	Duration, s
Tm	mean wave period, s
$t_{x,u}$	time taken for waves to travel the fetch length, s
t_m	thickness of the mattress, m
t	thickness of revetment, m
U_d	wind velocity, m/s
Us	sum of uplift forces, kN
U_w	wind speed, m/s

u	wind speed, m/s
u	shear velocity being proportional to the bottom velocity, m/s
u_1	upstream depth-averaged flow velocity, m/s
u_{cr}	scour depth, m
u_s	mean fluid velocity, m/s
$\bar{u}$	average velocity, m/s
u_{cr}	critical velocity, m/s
V	Mean velocity at the adjacent channel, m/s
V_{FA}	volume of falling apron per linear meter of protected length, m^3/m
$V_{thunderstorm}$	wind velocity, m/s
W	Weight of Individual Stone, Ib.wt
W	Width of barm, m
W_{II}	gravity force along the slope, kN
$W_{\llcorner}$	gravity force perpendicular to the slope, kN
WL	moisture content at liquid limit
Wp	moisture content at plastic limit
Y_{BL}	vertical distance between base level of falling apron at time of construction and deepest point of expected design scour hole, m
$y_{s,\ potrusion}$	Protrusion scour length, m
$y_{s,local}$	local scour depth, m
$y_{s,\ confluence}$	confluence scour depth, m
$y_{s,\ bend}$	Bend scour depth, m

Greek Symbols

τ	shear stress, N/m^2
β	circle angle, rad
Θ	angle between the porous flow direction and the horizontal, deg
α	angle of slope inclination, deg
α	slope angle of bank or structure, deg
ρ_{SAT}	specific mass of saturated soil , kg/m^3
Φ	angle of internal friction of soil, deg
τ_{cr}	critical shear stress, N/m^2
τ_o	shear stress exerted along the bank/bed boundaries, N/m^2
Ψ_{cr}	dimensionless critical Shields shear stress parameter for specific material
ρ_s	density of soil, kg/m^3
ρ_w	density of water, kg/m^3
Δ	relative density of submerged material = $(\rho_s-\rho_w)/\rho_w$
φ_{sc}	stability factor
ε_s	angle of repose considering the material specific internal friction, deg
θ	Slope of bank
Ψ	Shields Constant
ξs	angle of internal friction of material, deg
φ	stability factor for wave loads
Ψ	Permittivity, l/sec
η	material specific reduction factor

Part – I

General

CHAPTER ONE

Introduction

1.1 Importance

Rivers especially large rivers of Bangladesh are unique in behavior because of its dimensions, discharge, sediment characteristics and morpho-dynamic activities. There is active bank erosion almost in all major rivers in the country causing damage to valuable lands, settlements and infrastructures from year to year. Because of high density of population along the river bank a great number of people are also displaced due to this continuous bank erosion process. These poor displaced people migrate to near by towns and cities and live sub-human life in the slum areas. This has created a great national and social problem for the country. Bank protection and river training works are therefore, one of the prime necessities for poverty alleviation and national growth. The issue is the safety of lives, land and sustainability of the infrastructure against the forces acting in the rivers. The need of an up-to-date design guideline for this purpose can not be over emphased.

1.2 Design Guidelines and Manuals in Bangladesh

Prior to 1993 Bangladesh Water Development Board (BWDB) had no published Design Guidelines for River Training and Bank Protection. Up to that period BWDB Design Offices mainly used to follow the following publications for designing Bank Protection Works:

(1)River Training and Bank Protection, Bureau of Flood Control and Water Resources Department, Economic Commission for Asia and the Far East, (UN/ECAFE); Bangkok, Thailand, 1973.

(2)Manual on River Behaviour, Control and Training; D.V.Joglekar; Publication No.60, Central Board of Irrigation and Power, New Delhi, 1971.

3) River Behaviour, Management and Training; Publication No.204, Vol. I, Varma, C.V.J; Saxena, K.R. and Rao, M.K.,Central Board of Irrigation and Power, New Delhi, 1989.

BWDB published a design manual under the title, “Guide to Planning and Design of River Training and Bank Protection Works”, Design Circle-II, in 1993. Afterwards another Comprehensive Design Manual covering all aspects of hydraulic and hydrological design parameters including bank protection and river training works was published under the title, “Standard Design Manual” by Chief Engineer, Design, BWDB, 1995.

In 2001, Flood Action Plan 21 (FAP 21) published another guideline and manual under the title, “Guidelines and Design Manual for Standardized Bank Protection Structures”. [Bank Protection Pilot Project FAP 21, WARPO, December, 2001]. The manual documented some specific experience from bank protection works undertaken in Bangladesh and findings and recommendations from the pilot bank protection works undertaken in Brahmaputra-Jamuna River.

The present guideline does not replace the previous guidelines and manual rather updates the knowledge accumulated so far in Bangladesh systematically in a logical frame.

1.3 Scope of the Guideline

This guideline deals with the bank protection works in non tidal and non saline zone of Bangladesh. General principles, theories and typical structural solution developed from different projects like Bank Protection Pilot Project (FAP 21) and the River Training/ Active Flood Plain Management Pilot Project (FAP 22), River Bank Protection Project (RBPP), Jamuna Bridge, Jamuna Meghna River Erosion Mitigation Project (JMREMP) and Government of Bangladesh (GOB) funded projects for river bank protection structures in Bangladesh are described. The special feature is the experiences earned through different types of bank protection works in the major rivers of Bangladesh.

River training works are multi disciplinary. This guideline does no cover all related aspects of bank protection works. But it is stressed that the approach for decision making should be based on a policy analysis system taking into consideration of all the relevant aspects both of technical and non technical, social or political, national or international with their proper weightages. A mono-functional concept will conflict with the concept of sustainable development and will not bring the desired result in the short or long-term.

This guideline can be considered a step towards simplified and standardized design and implementation processes which helps for timely completion of bank protection structures within a restricted construction window and highly unpredictable river behavior scenarios encountered in Bangladesh.

1. 4 Layout of the Guideline

The intention of this Guideline is to guide the designer and engineers to a reliable design with correct choice within the available resources and environment of the country.

The guideline consists of three parts. Part-I contains the theoretical and general aspect for bank protection works. There are 5 (five) chapters in this part. Chapter 1 provides the importance, back ground and scope of application. Chapter 2 gives a brief description of types of rivers of Bangladesh and physical and geological aspects of the general planning environment. In Chapter 3 different erosion mechanism and protection structures are discussed. Chapter 4 discusses the geotechnical consideration. In Chapter 5, an outline of general planning and implementation aspect is given. Part-II deals with the design of bank protection works in Bangladesh. Part-III is concerned with the construction and maintenance of the protection works. Also there are two appendices to this guidelines, one of the appendices contains experience of major river bank protection works implemented in Bangladesh. In another appendix, design examples are provided using geobags for protection of erosion in Bangladesh at two main rivers completed under JMREMP.

1.5 The Guideline

Knowledge grows and develops with time. So is a design guideline. The knowledge presented in this Guidelines and Manual, hence, is not final. It needs updating from time to time as in done in other countries of the world.

CHAPTER TWO

River System and Estimation of Design Water Level and Discharge

2.1 Alluvial Rivers

Alluvial rivers are characterized by the fact that the alluvia on which the rivers flow, are built by rivers themselves. Because the beds and banks of alluvial rivers are readily erodible, they are less permanent than most other aspects of the landscape. This particular landform is dynamic, and it is readily and frequently affected by human activities. The main characteristics of these river reaches is the zigzag fashion in which they flow, called meandering. They meander freely from one bank to other and carry sediment which is similar to bed material. Material gets eroded constantly from the concave bank (outer edge) of the bend and gets deposited either on the convex side (inner edge) of the successive bends or between two successive bends to form a bar as shown in Fig 2.1.

One would assume a large river flowing in alluvium would maintain a relatively uniform morphology because its dimension should follow the rules of hydraulic geometry, and its gradient and pattern should reflect the type of sediment load and valley characteristics. The significant stream power exerted by these formidable fluvial systems should ensure that long reaches of alluvial channels maintain a characteristic and relatively uniform morphology. This simplistic assumption and the great variability of large alluvial rivers is an indication that hydraulics and hydrology are not always the dominant controls. In fact, the substantial energy of these mega river systems in many cases is inadequate to overcome accidents of geologic history and geologic controls. Large alluvial rivers appear to be sensitive to influences that can be relatively small. As a result, they are not monotonous in appearance. They frequently respond to factors that are not included in hydraulic models and sediment transport equations. Each alluvial river is unique; therefore, a geomorphic investigation is of prime importance if the channel hydraulics is to be improved, human facilities protected, and maintenance reduced to an economic level (Schumm and Winkley 1994).

Because of great range of alluvial river types, it is hard to generalize about their behaviour, and it has been found that it is not sufficient to document only the present conditions. It is apparent that prediction is improved if both past and present conditions are known because the record is extended by historical information.

Rivers on alluvial plain may be broadly classified into either (a) aggrading type and degrading type or (b) meandering type and braiding type. The classification depends on the size and the amount of sediment entering the river and its transport capacity for the sediment load. All these four types may be found in a single river from its uppermost point on alluvial plain to its outfall. A particular section of the river may also be aggrading, degrading, meandering or braiding at different times depending upon variation of sediment load and discharge. Among these types, the meandering type is the full and final stage of development.

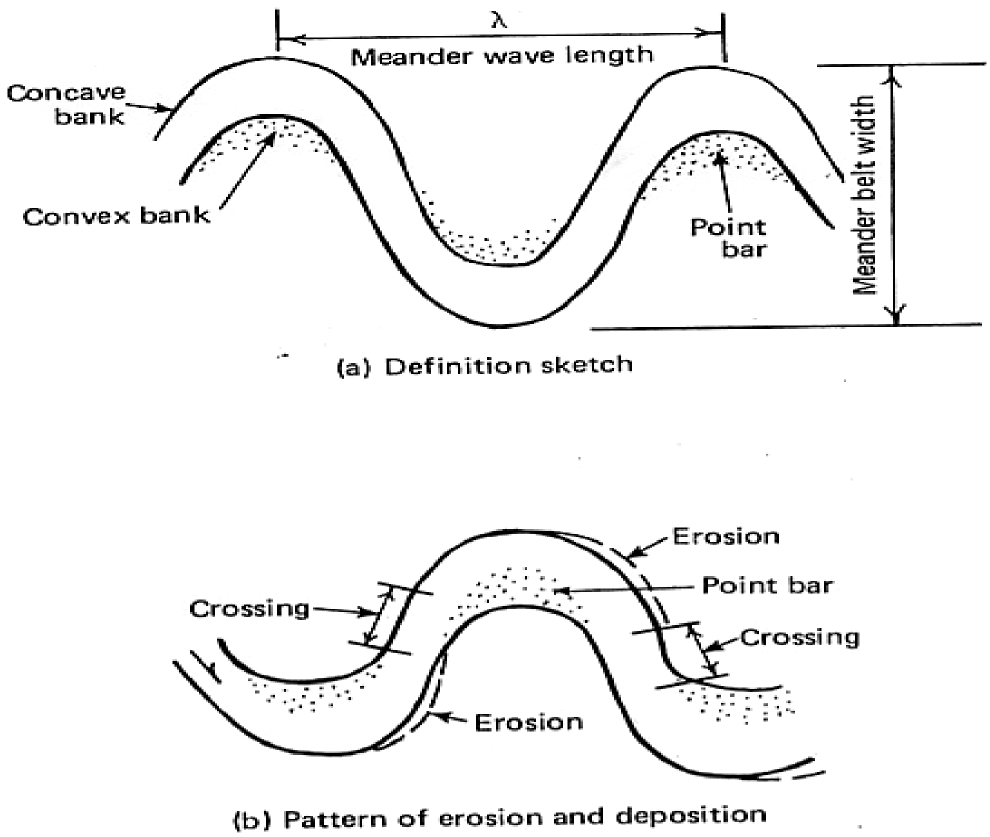


Fig 2.1 Meandering river (Peterson, 1986)

2.1.1 Aggrading Rivers

Owing to excessive sediment entering a river with sudden diminution of the slope on the plain, or owing to the extension of the delta at the river outfall, or to sudden onrush of sediment from a tributary, the river will build up its bed to a certain slope and is of aggrading type. This type of river, usually, has straight and wide reaches with shoals in the middle, which shift with floods, dividing the flow into a number of braided channels.

2.1.2 Degrading Rivers

The degrading type of river is in the process of loosing its bed gradually in the form of sediment load of the stream. If the river bed is constantly getting scoured, then it is known as degrading type. It may be found either above a cutoff or below a dam or a weir of a barrage, etc.

2.1.3 Meandering Rivers

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 planforms from those of straight and braided rivers. The term 'meandering', 'braided' and 'straight' are not mutually exclusive, because of different water stages the appearance of a river may change, depending on the number of channels visible at certain water levels.

Where a river has enough capacity to carry incoming sediment downstream without forming large deposits, the whole or part of it will be meandering type. Most of the sediment load carried by the river is brought to the sea.

When a river deviates from its axial path and a curvature is developed (either due to its own characteristics or due to the impressed external forces), the process moves downstream by building up shoals on the convex side by means of secondary currents (Fig 2.2). The formation of shoals on the convex side, results in further shifting of the outer bank by eroding on the concave side.

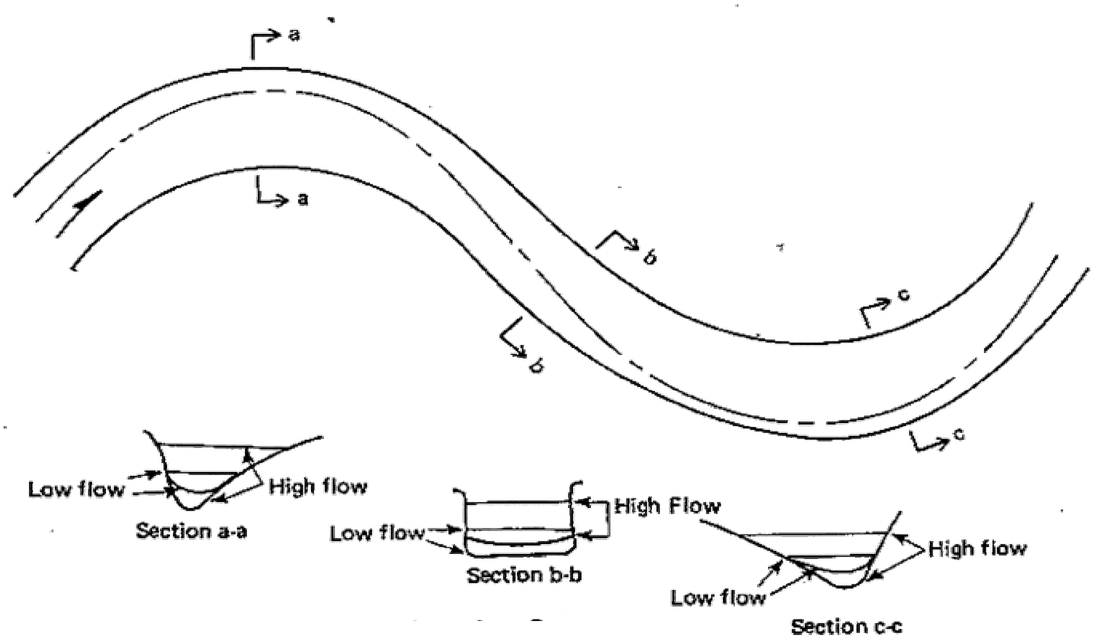


Fig 2.2 Meandering process at bends (Peterson, 1986)

Formation of successive bends of reverse order may lead to the formation of a complete S curve called meander. When consecutive curves of reverse order connected with short straight reaches (called crossings) are developed in a river reach, the river is stated to be a meandering river.

2.1.3.1 Causes of Meandering

The exact condition under which a stream meanders and the exact mechanism of such process are not yet known completely. The latest and widely accepted theory behind meandering is based upon the extra turbulence generated by the excess of river sediment during floods. It has been established that when the silt charge is in excess of the quantity required for stability, the river starts building up its slope by depositing the silt on its bed.

This increase in slope tends to increase, in turn, the width of the channel if the banks are not resistant. Only a slight deviation from uniform axial flow is then necessary to cause more flow towards one bank than towards the other. Additional flow is immediately attracted towards the former bank, leading to shoaling along the latter, accentuating towards the curvature of flow and producing finally, meander in its wake.

Four variables, governing the meandering process are: (i) valley slope, (ii) silt grade and silt charge, (iii) discharge, and (iv) bed and side materials and their susceptibility to erosion. All these factors considerably affect the meandering patterns, and all of them are interdependent. On the basis of this theory and experiments, various empirical formulas have been developed for connecting different meander parameters; but much remains to be learnt about the meander process (Leopold and Wolman, 1959; Lame, 1953; Chang, 1989; Hossain, 1989).

2.1.3.2 Meandering Parameters

Meander Length (M_L). It is the axial length of one meander, i.e. the tangential distance between the corresponding points of a meander (Fig 2.1a).

Meander Belt or Meander Width (M_B): It is the distance between the outer edges of clockwise and anti-clockwise loops of the meander.

Meander Ratio: It is the ratio of meander belt to meander length, i.e., M_B / M_L.

Sinuosity: It is the ratio of the length along the channel (or thalweg length) to the direct axial length of the river reach, (or valley length).

Crossing or Cross-over: The short straight reaches of the river, connecting two consecutive clockwise and anti-clockwise loops, are called crossing or cross-over.

Tortuosity: Jogleker (1971) defined tortuosity as

$$\text{Tortuosity} = \frac{(\text{Thalweg length} - \text{Valley length})}{\text{Valley length}}$$

The term meandering is used for channels or rivers with a sinuosity larger than 1.25 or 1.5 (FAP21, 2001). Leopold and Wolman (1957) classified stream with sinuosity greater than 1.5 as meandering stream.

2.1.4 Braiding and Anabranching Rivers

The multiple channels of rivers like Jamuna are partly braiding and partly anabranching (Fig 2.3). 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. 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. Each anabranch is a distinct and rather permanent channel with banklines, whereas braid 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 flood plain (FAP21/22 2001a).

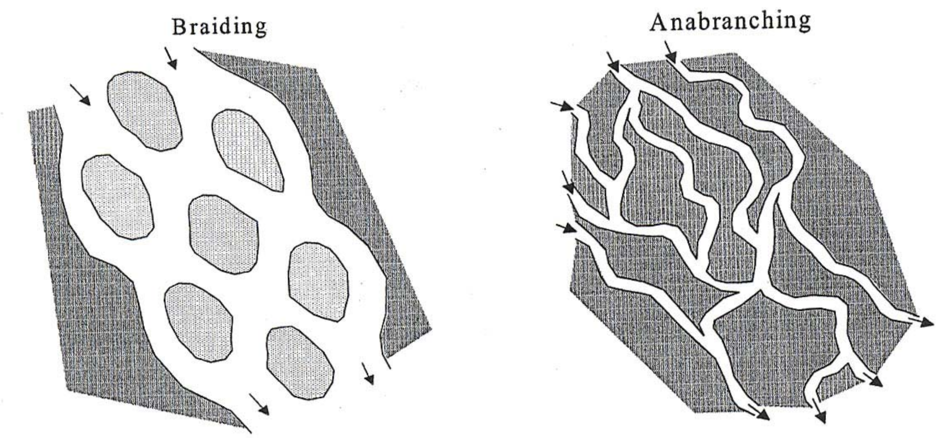


Fig 2.3 Braided and anabranched rivers (schematic) (FAP21/22 2001a)

A typical cross section of a braiding river is shown in Fig 2.4; the active flood plain 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 areas 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 bankful discharge is between one and one and a half years. Chars with a crest level above the bank full discharge are anticipated as quite stable compared to lower chars, which are temporary features only.

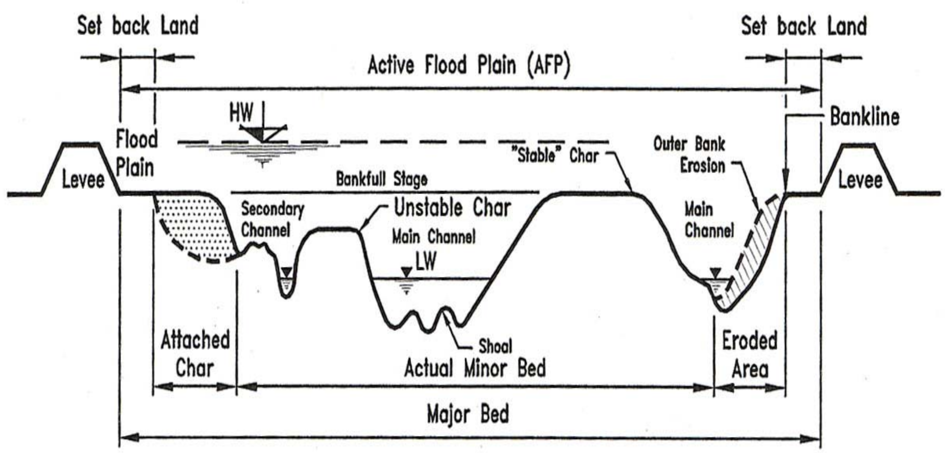


Fig 2.4 Typical cross section of a braiding river (FAP21/22 2001a)

2.2 Rivers of Bangladesh

2.2.1 Main Rivers and Hydrological Aspects

Bangladesh is located at the lower part of the basins of three mighty rivers, the Ganges, the Brahmaputra and the Meghna. The total catchment area of these three rivers stands at 1.72 million sq.km covering areas of China, India, Nepal, Bhutan and Bangladesh of which only about 8% lie within Bangladesh (Fig 2.5). The country is criss-crossed by more than 230 rivers, most of which are either tributary or distributary to the three major rivers (Fig 2.6). There are 57 rivers which originate outside the boundary of Bangladesh. The total length of the river course is approximately 24,000 km and cover

9,770 km^2 or 7% of the country. The annual volume of flow past Baruria just below the confluence of Brahmaputra and Ganges is 795,000 million m^3.

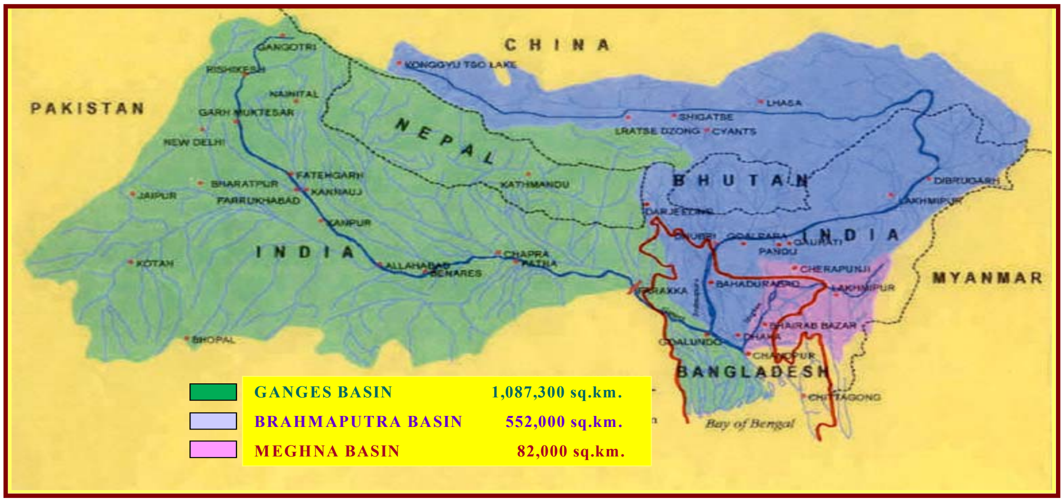


Fig 2.5 Ganges, Brahmaputra and Meghna Basins

The Brahmaputra-Jamuna River draining the northern and eastern slopes of the Himalayas is 2900 km long and has a drainage area of 573,500 km^2. In the Bangladesh reach (length 240 km) the river has several right bank tributaries, the Teesta, the Dharla, the Dudhkumar, etc and two left bank distributaries – the Old Brahmaputra and the Dhaleswary Rivers. It is a wandering braided river with an average bankful width of about 11 km. The channel has been widening, increasing from an average of 6.2 km in 1834 to 10.6 km in 1992 (FAP16 1995). Having an average annual discharge of 19,600 m^3/sec, the river drains an estimated 620×10^9 m^3 of water annually to the Bay of Bengal. The discharge varies from a minimum of 3,000 m^3/sec to a maximum of 100,000 m^3/sec, with bankful discharge of approximately 48,000 m^3/sec. It has an average surface slope of 7 cm/km (Hossain 1992).

The Ganges River draining the south slope of Himalayas has a drainage area of 1,090,000 km^2 and a length of 2200 km. It is a wide meandering river with a bankful width of about 5 km. In the Bangladesh reach (length about 220 km), a left bank tributary, named the Mahananda, joins the river upstream of the Hardinge Bridge and a right bank distributary, the Gorai, carries part of the high stage Ganges flow to the Bay of Bengal. The annual average discharge of the river is about 11600 m^3/sec and drains 366×10^9 m^3 of water annually to the Bay of Bengal. The discharge varies from a minimum of 1,000 m^3/sec to a maximum of 70,000 m^3/sec, with a dominant discharge of about 38,000 m^3/sec. The dry period discharge decreased from 2,250 m^3/sec before 1975 to only 650 m^3/sec after 1975 (RSP 1994). The water surface slope of the river is about 5 cm/km. However the dry period discharges is often much lower than 650 m^3/sec (Hossain 1989).

The Upper Meghna River originates in the Shillong Plateau and foothills. It is a canaliform type of meandering river and locally anabranched. It is relatively a small river having a bankful width of about 1 km. The river has a catchment area of about 77,000 km^2 and a length of about 900 km. The river drains 151×10^9 m^3 of water annually to the Bay of Bengal. The average annual discharge of the river is about 4,800 m^3/sec and dominant discharge is about 9,500 m^3/sec. The lower reach of the river becomes tidal during December-April period with negligible residual flow and reaches a maximum discharge of about 20,000 m^3/sec during monsoon.

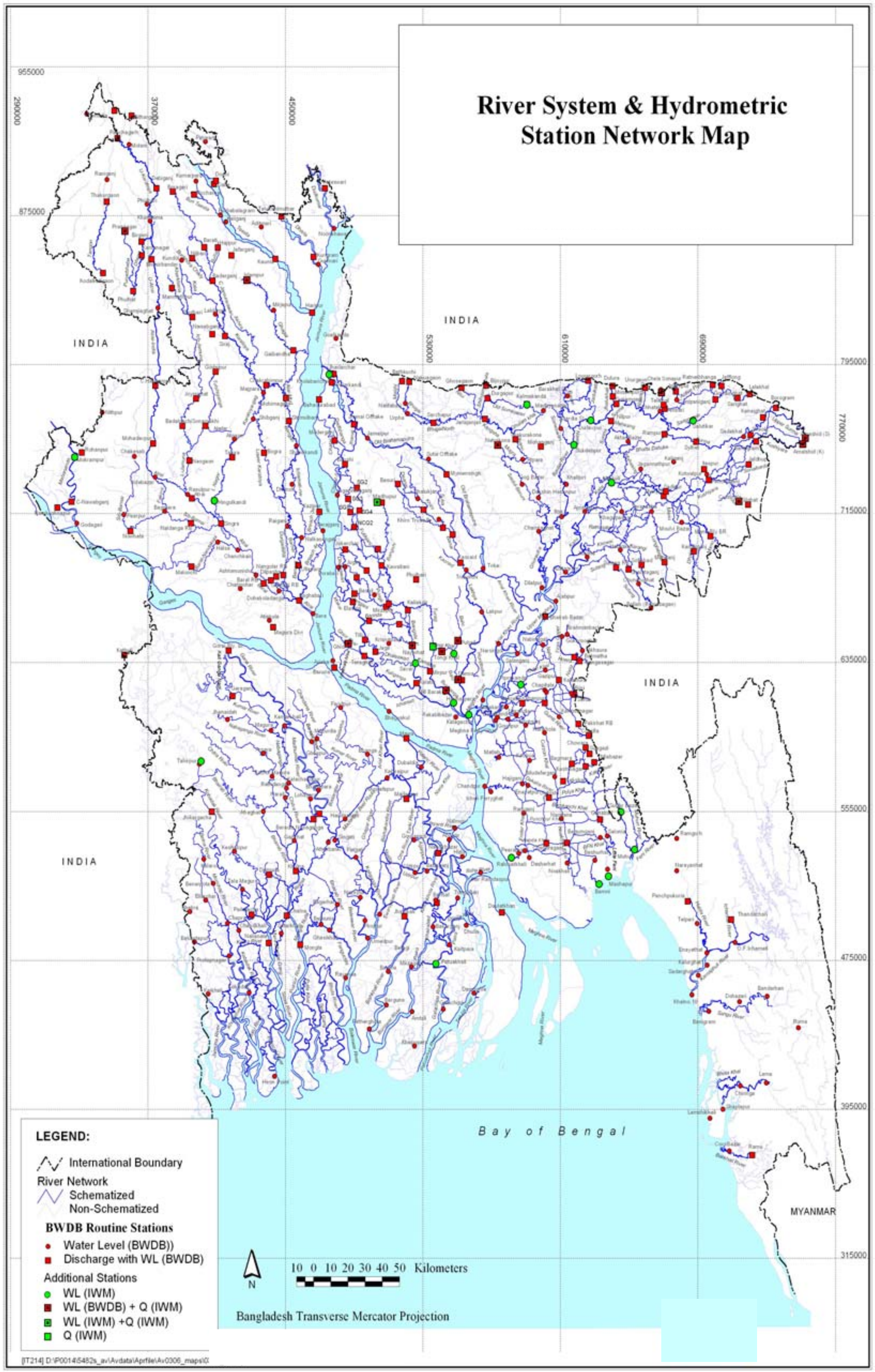


Fig 2.6 Major and medium rivers of Bangladesh

The Ganges and the Brahmaputra-Jamuna River meet near Aricha forming the Padma River, which flows southeastward until it reaches the Upper Meghna River near Chandpur. It is more or less a straight reach of about 120 km length. Prior to avulsion of the Brahmaputra River, it was just the lower reach of the Ganges River debouching into the Bay of Bengal through Arial Khan River. After the historic change of the Brahmaputra River, the river, swollen to a big size, initially followed the same Arial Khan course and later made a rapid shift to the northeast joining the Upper Meghna River near Chandpur. The discharge variation at Baruria is from a minimum of about 5,000 m^3/sec to a maximum of about 120,000 m^3/sec.

The Lower Meghna River is a tidal reach carrying almost the entire fluvial discharge of the Ganges, the Brahmaputra-Jamuna and the Upper Meghna River. The net discharge through this river varies from 10,000 m^3/sec in the dry season to 160,000 m^3/sec in the wet season.

Surma-Kushiyara Rivers located in the northeast region of Bangladesh. The Surma-Kushiyara Rivers are formed by bifurcation of the Barak River which is flowing from India, enters into Bangladesh at Amalshid where the flow is divided into two branches known as Surma and Kushiyara Rivers. These rivers suffer from an unequal distribution of inflow of Barak river, Surma receiving about 40% of flood flow and no flow in dry season (Bari and Marchand, 2006). In the dry season, there is a remarkable difference between the Surma River reach before Lubachhara River and the reach after Lubachhara. The Lubachhara River that comes from Cachar Hills of Assam and other tributaries (Sari, Goyain, Piyain, Dhalai Rivers, etc) feed Surma River. On the other hand, the Kushiyara River is getting more water from the Barak River due to its straight intake. Moreover, about 60-70 km downstream from the bifurcation there are several tributaries (e.g. Sonai Bardhal, Juri, Manu Rivers, etc) of Kushiyara. The rivers experience flash flood in pre-monsoon period and river floods in monsoon. The average recorded daily flow of Kushiyara at Sheola (Bangladesh gauging station) is 628 m^3/sec, minimum average flow (January to March) is 90 m^3/sec and average monsoon (May to October) flow is 1100 m^3/sec. The average daily recorded flow of Surma at Kanaighat is about 500 m^3/sec; the average monsoon (June to September) flow is 1200 m^3/sec.

The Teesta rises in the very high Sikkim Himalaya. Several generally southerly flowing tributaries draining the Sikkim mountains and two east and west flowing tributaries forming the Sikkim-Darjeeling border meet on the border at about 15 km east-north-east of Darjeeling City to form the main Teesta in India. In its upper reaches the Teesta is a steep and swiftly flowing river and in the lower reaches at about 25 km from the Sikkim-Darjeeling border, as the Teesta meets the flatter piedmont terrain, the river widens considerably to form a braided channel of 2 km to 4 km width. From this point, the Teesta follows a south-easterly and comparatively straight course to meet Bangladesh border 100 km downstream and the Brahmaputra-Jamuna River further 120 km downstream. Catchments area of the river up to Dalia is about 10,100 km^2. Average daily discharge at Dalia is about 836 m^3/sec, average discharge during five minimum flow months (December-April) is about 200 m^3/sec and that during four wet months (June-September) is about 1800 m^3/sec.

The Dharla is a western tributary of the Brahmaputra-Jamuna which rises in the Himalayas in the southeast of Sikkim and southwest of Bhutan. Northernmost point of its catchment is in Sikkim at about 25 km east of Gangtok, where the Jaldhaka, a major tributary of Dharla, originates. The Jaldhaka flows southward from this point for about 10 km into Bhutan and after a further 15 km as a deep gorge it becomes the boundary for 20 km between Bhutan and the Darjeeling District of India. The Jaldhaka flows past Mathabhanga Town and is then joined by the Dharla, a much smaller tributary from the west. In Bangladesh the length of the river is about 20 km from its entry to outfall at Brahmaputra near Kurigram. Total catchment area of the river is about 5220 km^2. Average daily discharge of the river is about 326 m^3/sec, average flow during the five low flow months (December-April) is about 90 m^3/sec and that during five wet months (June-October) is about 600 m^3/sec.

The Dudhkumar is a tributary of the Brahmaputra rises in the tract of Tibet. It passes through Bhutan, Sikkim and through the North Bengal Plains (India) before it enters Bangladesh. The upper reaches of the river in India is called Torsa (Raidak). Length of the river from source to Bangladesh border is 190 km and that within Bangladesh up to outfall is 30 km. Total catchment area up to Bangladesh gauging station is 11,230 km^2. The catchment area within Bangladesh is about 398 km^2. The average daily flow at Pateswary Gauging Station is about 500 m^3/sec, average daily flow in the four dry months (January-April) is 118 m^3/sec and that during the five wet months (June-October) is 975 m^3/sec.

The Old Brahmaputra River appears to be going through a secular period of aggradations. The river is at present reduced to a spill channel of the Brahmaputra-Jamuna River, active only during high river stage. Its hydrological regime is flashier than perennial. The average annual discharge of the river is showing a decreasing trend, from being 800 m^3/sec in 1964 to 500 m^3/sec in 1991 (RSP 1994). A study of the satellite imageries revealed that its offtake was nearly closed during 1973-82 period; became wider again during 1983-89 period and nearly closed again after 1990. The offtake is affected by the overall westward migration or channel shifting of Brahmaputra-Jamuna River.

The Dhaleswary River offtakes at two locations, the downstream offtake being more active than the upstream offtake. The upstream offtake has been closed by the Jamuna Bridge construction activities, though a spill channel has developed just downstream of the bridge site. The annual average discharge of the Dhaleswary River reduced from about 1,750 m^3/sec in 1969 to 600 m^3/sec in 1987 (RSP 1994). This range of discharge represents some 7 % and 4% of Jamuna River discharge.

The Gorai is a right bank distributary of the Ganges River debouching independently into the Bay of Bengal after capturing some local flows. The lower reach of the river is tide dominated. Historic records indicate that the river was once a major course of the Ganges River. The river is fast loosing its conveyance function. The average discharge of the river is only about 1,400 m^3/sec (1994) representing some 13% of Ganges River flow. Even this represents only the wet months from June to October. During the rest of the year the offtake remains dry.

The Arial Khan is a right bank distributary of the Padma River. The river discharges directly into the Bay of Bengal. The entire reach of the river is tide dominated. The river has an average annual discharge of 2,600 m^3/sec representing about 9% of Padma River flow. The mean annual flow appeared to be increasing at a rate of some 42 m^3/sec (RSP 1994). It has more than one offtake. A comparison of the images between 1960 and 1989 around the offtake of the river with the Padma River reveals that Arial Khan continued right bank erosion of the Padma with consequent shift of the offtake location. The sinuosity in the upper reach of the river appeared to have decreased from 1.8 to 1.5 in 16 years (1973-89) (Hossain et al. 2007).

A comparison of available values for the average discharge, bankful discharge, dominant discharge (that carries most of the sediment transport of the river), slope and sediment grain size for selected rivers are shown below from different sources in Table 2.1.

Table 2.1 Compiled values of discharge and other parameters for different rivers
Source: Haskoning (1992), Thorne et al (1993), Halcrow (1993), FAP 24 and others

River	Type/C. area km^2	Length km Width km/ m	Discharge m^3/s	Grain size mm & slope	Comments
Brahmaputra-Jamuna	wandering braided & anabranched 573,500	L_{total}= 2900 L_{BD}= 240 W_{bank} =11km	Q ≈3000-100,000 Q_{avg}= 19,600 Q_{dom}= 38,000 Q_{bank}= 48,000	$d_{bed-median}$ = 0.165-0.22 S=0.00007	width: 6.2 km in 1830 & 10.6 km in 1992
Ganges	wide meandering 1,090,000	L_{total}= 2200 L_{BD}= 220 W_{bank} = 5km	Q ≈ 1000-70,000 Q_{avg}= 11,000 Q_{dom}= 38,000 Q_{bank}= 40,000-45,000	$d_{bed-median}$ = 0.12 S=0.00005	river swings in active corridor; Q_{dry} =2250 m^3/s in 1975 but later as low as 650 m^3/s
Upper Meghna	canaliform type meandering, locally anabranched77,000	L_{total}= 900 W_{bank} = 1km	Q_{max} ≈ 30,000 Q_{avg}= 4800 Q_{dom}= 9500 Q_{bank}= ?	$d_{bed-median}$ = 0.14 S=0.00002	originated in Shillong plateau & foothills; becomes tidal in the dry season
Padma	meandering & braiding combination 1163	L_{total}= 120 W ≈ 3-15km	Q ≈ 5000-120,000 Q_{avg}= 28,000	$d_{bed-median}$ = 0.09-0.14 S=0.00009	wandering river swings within active corridor; weakly tidal
Lower Meghna	1595	L_{total}= 180 W= 13 km	Q ≈ 10,000-160,000		tidal
Teesta	braided 10,100 (up to Dhalia)	L_{BD}= 120 W ≈ 2-4km	Q ≈ 200-1800 Q_{avg}= 836 (in Dhalia) Q_{dom}= 2270	$d_{bed-median}$ = 0.08-0.35 S=0.00047 (u/s barrage) 0.08-0.25 S=0.00031 (d/s barrage)	
Kushiara (Kalni)	meandering 26,000 in India, A_{BD} = 10,945	L_{BD}= 229 W=157m	Q ≈ 90-1100 Q_{avg}= 628 (at Sheola)		in India it is called Barak
Surma	meandering 700 in India, A_{BD} =7476	L_{BD}= 245 W=90m	Q_{max} ≈ 1200 Q_{avg}= 500 (at Kanairghat)		
Dharla	5220 (total)	L_{BD}= 20 W ≈ 2-4km	Q ≈ 90-600 Q_{avg}= 326		
Dudhkumar	11,230 (total) 398 (in BD)	L_{BD}= 30 L_{India}= 190 W ≈ 2-4km	Q ≈ 118-975 Q_{avg}= 500		It was Q_{avg} = 800 m^3/s in 1964 & 500 m^3/s in 1991
Daleshwari	7253	L_{total}= 168 W= 300m	Q ≈ 21-2330 Q_{avg}= 600 Q_{bank}= 2100	$d_{bed-median}$ = 0.18 S=0.000045	lower offtake is more active
Gorai	4568	L_{total}= 90 W= 400m	Q ≈ 35-4000 Q_{avg}= 1400 (wet months)	$d_{bed-median}$ = 0.179 S=0.00004	lower reach is tidal
Arial Khan	1438	L_{total}= 163 W= 300m	Q ≈ 47-4000 Q_{avg}= 2600	$d_{bed-median}$ = 0.147	entirely tidal
Atri Lower	2770	L_{BD}= 200 W= 120m	Q ≈ 172-2958		
Gomoty	41	L_{BD}= 130 W=122m	Q ≈ 2-985		
Mahananda Lower in Chapai N.	1300	L_{BD}=67 W=300 m	Q=14-2297		
Karnafuli	1296	L_{BD}= 160 W=150m	Q ≈ 1155-10761		
Buriganga	253	L_{BD}= 45 W=265m	Q ≈ 50-1500		
Sitalakhya	3803	L_{BD}= 73 W=273m	Q ≈ 195-2742		
Matamohuri	429	L_{BD}= 120 W=100m	Q ≈ 3-750		
Little Feni	561	L_{BD}= 80 W=180m	Q ≈ 112-468		
Ichamoti in Dinajpur	270	L_{BD}=82 W=50m	Q=0-75		
Bangali	1100	L_{BD}= 148 W=150m	Q ≈ 8-3750		
Kobadak in Chuadanga	800	L_{BD}= 180 W=150m	Q ≈ 2-80		
Dhakatia	486	L_{BD}= 110 W=180m	Q ≈ 10-600		

2.2.2 River Network and Morphology

The main rivers of Bangladesh are the Ganges and the Brahmaputra originating in the south and north eastern Himalaya respectively and the Meghna, draining the Sylhet basin in the north-eastern part of Bangladesh. Together with numerous tributaries and distributaries a dense network of rivers is formed, 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 (Fig 2.5). The rivers have changed their courses frequently in the past, the Brahmaputra switched from its eastward course in favor 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 of Bengal through the Arial Khan River, but after capture of the Brahmaputra-Jamuna River flow it shifted north-eastward joining the Upper Meghna (Fig. 2.7).

Several years map compilation by ISPAN (1993) indicates that in about 158 years (1834-1992), the rivers of the entire Bangladesh migrated westward by bank-erosion with an average rate of some 50 m per year. Apart from the secular westward migration, erosion occurs at different rates in different hierarchies of channels (Bristow 1987; Halcrow 1993a; Thorne et al. 1993; RRI-CNR-DELFT 1993).The erosion is mostly concentrated in outer-bank embayment. Satellite imagery interpretations by Klaassen and Masselink (1992) showed that bank erosion in curved channels occurs at various rates ranging from 0 to 500 m with a maximum of some 1000 m/year and they mostly occur perpendicular to the primary flow direction (in the east-west direction). Studies by ISPAN (1993) indicate that the Brahmaputra-Jamuna River widened in the 1834-1992 period from 6.2 to 10.6 km representing a widening of some 27 m/year on average. In 19 years period from 1973-1992 the rate of erosion/widening accelerated to some 140 m/year on average. Bhuiya and Rahman (1988) reached the same conclusion of secular widening of the river. They found the widening at a rate of 172 m/year between 1972 and 1986.

Recent analysis of the satellite images by CEGIS (CEGIS, April 2007) for the last few decades shows that the river is widening along both banks. During the last three decades (1973-2007) the net erosion along the 240 km reach of the Jamuna River were about 77,480 ha. The rate of widening of Jamuna River declined from 150 m/year in 1970s and 1980s to 48 m/year in last 14 years (late 1990s to 2006).

Map compilation by ISPAN (1993) shows that the Ganges river has not shifted significantly over the past 200 years. Similarly very insignificant widening took place between 1984 and 1993. 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 embayment and nodal points along the river have been observed, in between which the river wanders. The erosion rate of the Ganges is quite high in certain reaches 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.

Prior to the avulsion of the Brahmaputra River, the Ganges River was just the lower reach of the Ganges debouching into the Bay of Bengal through the Arial Khan River. After the historic change of the Brahmaputra River, the river, swollen to a big size, initially followed the same Arial Khan course and later made a rapid shift to the northeast joining the Upper-Meghna River near Chandpur (Fig 2.8). The Padma River has roughly a straight course in the upper reaches and a double-threaded braided lower reach. Study by ISPAN (1993c) indicates that the river widened considerably. In the lower reaches some 46% widening took place between 1984 and 1993, the middle reach widened by 21% during the same period. The upper reach remained relatively stable, which may be attributed to a stable hard clay bank on its left bank near Baruria. The right bank of the middle and the lower reaches, on the other hand, is experiencing considerable erosion with some 200 m/year in the lower reach to some 110 m/year in the middle reach. At the left bank the erosion is some 50 m/year (ISPAN 1993).

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 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.

For the foregoing deliberation, it can be stated that the bank erosion rates of the three main rivers are very similar. However, for 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. For the Brahmaputra/ Jamuna the flow attacks any of the banks and new channel courses outside the active floodplain are created frequently.

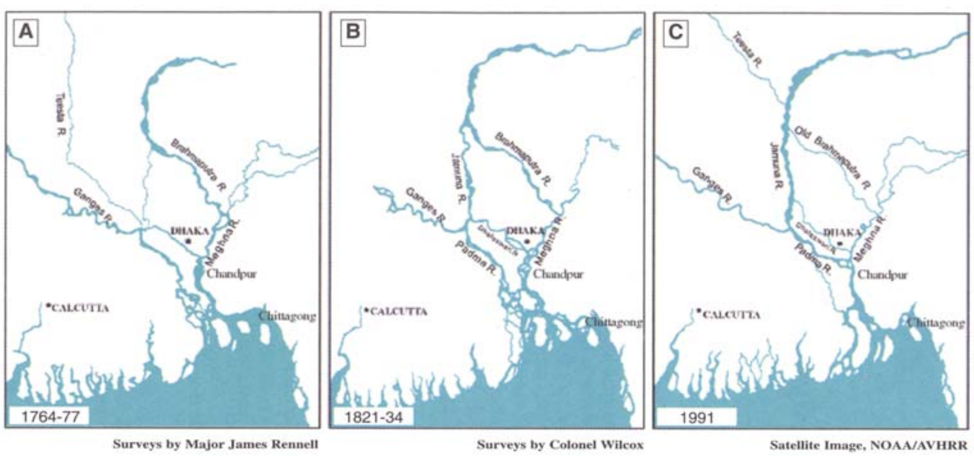


Fig 2.7 Historic developments of rivers in Bangladesh

2.2.3 Sediment Transport

The rivers of Bangladesh are characterized by a fine sedimentary environment. The consequence is that the threshold velocity for sediment mobility is low, about 0.2 m/s (Hjulstrom, 1935). Due to this low threshold velocity, river are highly mobile with continuous reworking and deformation of their beds and banks transporting huge quantities of sediment.

Among the three major rivers, Brahmaputra River has the highest sediment caliber and transports the largest sediment load. The median bed-material diameter varies from 165 μm near Aricha to 220 μm near Chilmari. The mean annual suspended sediment transport estimates of the river varies from 387 to 815 million tons ((MPO-HARZA

1987, Hossain 1992). The estimated sediment load is in the order of 10 million metric tons/day in flood (Coleman 1968, Hossain 1992).

The Ganges River has a more fine-sedimentary environment than the Jamuna. The median bed material diameter is about 120 µm (NEDECO 1967).The mean annual suspended sediment transport varies from 196 (CBJET 1991) to 549 million tons as reported by the RSP (1994). Hossain (1992) estimated the annual average sediment loads to vary from 350 million tons to 650 million tons.

Padma River bed material sediment diameter varies from 90 µm in the lower reaches to 140 µm in the upper reaches. Carrying the combined flow of the Ganges and Brahmaputra Rivers, the mean annual suspended sediment transport estimate varies from 563 (MPO-HARZA 1987) to 894 million tons by the RSP (1994).

The Upper-Meghna River has a median diameter of 140 µm and it transports about 13 million tons of suspended sediment annually. Among the distributaries, the Dhaleswary River transports about 19 million tons and the Gorai River transports some 47 million tons annually.

One important aspect of the sedimentology of Bangladesh Rivers is the overabundance of silty materials. Thorne et al. (1993) indicated that the Jamuna River silt transport occupies 75% of the total transport. The RSP (1994) estimates for all the major rivers indicate that the silt transport is about 59 to 84% of the total transport.

2.3 Estimation of Design Discharge and Water Level

Estimation of both flood discharges and high water levels are necessary for bank protection design. Careful estimation of discharge and water level is important for all sites with erodible banks. This section describes the methods of assessing flood discharge and water level at the site under consideration. The design discharge and water level are determined for selected probability of exceedance or return period.

Choice of Return Period: The design discharge and water level arising from floods should be selected after due consideration of the following:

- The maximum historical discharge as recorded at the site, or as calculated on the basis of recorded water level at the site, or as calculated on the basis of measured discharge at other points on the river from which corresponding site discharge can reasonably be inferred;

- the discharge derived from a frequency analysis using a probability of exceedance or return period which is appropriate to the importance and value of the protection work.

- The maximum historical water level as recorded at the site, or as inferred from observed or recorded water level at other points on the river from which level can reasonably be transferred to the site in question;

- the water level derived from a frequency analysis using a probability of exceedance or return period which is appropriate to the importance and value of the protection work.

In estimating high flows, primary reliance should be placed on careful field investigations, local enquiries and searches of historical records. Data so obtained should be compared with recorded data for hydrometric stations, and supplemented by analytical procedure using stage-discharge curves: At most hydrometric gauging stations reasonably stable relationship exists between water level and discharge. At some sites, however, the stage discharge curve may be quite unstable because of aggradation or degradation at channel bed or backwater effect from downstream, and may change drastically during major floods. A persistent trend of rising or lowering of curve indicates progressive channel aggradation or degradation. The stage corresponding to design flood which exceeds any recorded flow obtained by extrapolating the stage-discharge relationships, cautions must be made where the design flow involves substantial overbank flow or where the river bed is liable to scour. Neill (1973) suggests that a synthetic stage-discharge curve at any point on a stream can be constructed by applying the slope area method to calculate the discharge at successive stages.

The most commonly used method for estimating design discharge and water level examines the observed discharge and water level to arrive at suitable estimates. The method, known as frequency analysis, is founded on statistical analyses of discharge and water level records. For locations where records of stream flows are available, or where flows from another basin can be transported to the design location, design flood magnitude and water level can be estimated directly from those records by means of frequency analysis.

2.4 Frequency Analysis

Frequency of a hydrologic event, such as the annual peak flow is the probability that a value will be equaled or exceeded in any year. This is more appropriately called the exceedance probability*, P(F).* The reciprocal of the exceedance probability is the return period T in years, i.e., $T = {}^{1}\!/_{P(F)}$. The length of record should be sufficient to justify extrapolating the frequency relationship. For example, it might be reasonable to estimate a 50-year flood on the basis of a 30-year record, but to estimate a 100-year flood on the basis of a 10-year record would normally be absurd (Neill 1973). Viessman and Lewis (1996) noted that as a general rule, frequency analysis is cautioned when working with shorter records and estimating frequencies of hydrologic events greater than twice the record length.

Frequency analysis can be conducted in two ways: one is the analytical approach and the other is the graphical technique in which flood magnitudes are usually plotted against probability of exceedance.

Here in the following sections, procedures are given mostly for discharge frequency analysis; the similar procedures can also be followed for water level frequency analysis.

2.4.1 Analytical Frequency Analysis

Analytical frequency analysis is based on fitting theoretical probability distributions to given data. Numerous distributions have been suggested on the basis of their ability to "fit" the plotted data from streams (Linsley et al. 1988). The Log-Pearson Type III (LP3) has been adopted for use in the United States federal agencies for flood analysis. The first asymptotic distribution of extreme values (EV1), commonly called Gumbel Distribution has been widely used and is recommended in the United Kingdom. EV1 Distribution was found to fit peak flow data for several rivers in Bangladesh (Bari and Saleque 1992; Ahmed and Rahman 1986, Haque et al. 1998).

Extreme Value Distributions: Distributions of the extreme values selected from sets of samples of any probability distribution converge to any one of three forms of Extreme Value Distributions, called Type I, II, and III, respectively, when the number of selected extreme values is large. The three limiting forms are special cases of a single distribution called Generalized Extreme Value (GEV) Distribution. (Chow et al. 1988). The cumulative distribution function for the GEV is

$$F(x) = \exp\left[-\left(1-\kappa\frac{x-u}{\alpha}\right)^{1/\kappa}\right] \tag{2.1}$$

where κ, u, and α are parameters to be determined. For EVI Distribution x is unbounded, while for EVII, *x* is bounded from below, and for EVIII, x is bounded from above. The EVI and EVII Distributions are also known as the Gumbel and Frechet Distributions, respectively.

The Extreme Value Type I (EVI) cumulative distribution function is

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\alpha}\right)\right] \qquad -\propto \le x \le \propto \tag{2.2}$$

The parameters are estimated by

$$\alpha = \frac{\sqrt{6}}{\pi}s \quad \text{and} \quad u = \bar{x} - 0.5772\alpha \tag{2.3}$$

Eq (2.2) can be expressed as

$$F(x) = e^{-e^{-y}} \tag{2.4}$$

where y is the reduced variate defined as

$$y = \frac{x-u}{\alpha} \tag{2.5}$$

Solving Eq (2.4) for y:

$$y = -\ln\left[\ln\left(\frac{1}{F(x)}\right)\right] \tag{2.6}$$

Noting that the probability of occurrence of an event $x \ge x_T$ is the inverse of its return period T, we can write

$$\frac{1}{T} = P(x \ge x_T) = 1 - P(x \le x_T) = 1 - F(x_T)$$

so $$F(x_T) = 1 - \frac{1}{T}$$

and substituting for $F(x_T)$ into Eq (2.6)

$$y_T = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \tag{2.7}$$

For a given return period x_T is related to y_T by Eq (2.5), or

$$x_T = u + \alpha y_T \tag{2.8}$$

Example 1

Using the EVI Distribution, a model is developed for frequency analysis of the annual peak flow data of Old Brahmaputra River at Mymensingh for 5, 10, 25 and 50 years return period peak flows are calculated.

Annual peak discharges (m^3/s) of the Old Brahmaputra River at Mymensingh for the period from 1964-98

Year	Peak flow	Year	Peak flow	Year	Peak flow
1964	2830	1978	2770	1989	2180
1965	3230	1979	2630	1990	2060
1966	3490	1980	3340	1991	2900
1967	3000	1981	2690	1992	1490
1968	2810	1982	2470	1993	2060
1969	2770	1983	2370	1994	1065
1970	3250	1984	4780	1995	3187
1974	3820	1985	3070	1996	2369
1975	3060	1986	1930	1997	1973
1976	3210	1987	3230	1998	3267
1977	3550	1988	4910		
Sample Size		n = 32			
Max =	4910	Ave, $\bar{x}$ =	2867.53		
Min =	1065	Std, s =	804.54		
Skew, C_s =	0.372				

Note that data for 1971, 72 and 73 are missing. When a fairly long record has a short gap, it may be justifiable to estimate the missing data by correlation with a nearby station; otherwise it is preferable to consolidate the various recorded sequences as if they formed a continuous record (Neill 1973). The latter approach is used in this example.

For the given data $\bar{x} = 2867.53$ and s = 804.54. Substituting in Eq (2.3) yields

$$\alpha = \frac{\sqrt{6} \times 804.54}{\pi} = 627.62$$

and $u = \bar{x} - 0.5772\alpha = 2505.27$

The probability model is $F(x) = \exp\left[-\exp\left(\frac{x - 2505.27}{626.62}\right)\right]$

To determine the values of x_T for various values of *T*, it is convenient to use the reduced variate y_T.

For *T* = 5 years, Eq (2.7) gives $y_5 = -\ln\left[\ln\left(\frac{5}{5-1}\right)\right] = 1.5$

and Eq (2.8) yields x_5 = 2505.27 + 627.62×1.5 = 3446.7 m^3/s.

Similarly for other values of *T*, y_T and x_T values are found as follows:

T=10 years, $y_T = 2.25$, $x_T = 3918$ m^3/s

T=25 years, $y_T = 3.20$, $x_T = 4513$ m^3/s

T=50 years, $y_T = 3.90$, $x_T = 4954$ m^3/s

Frequency Analysis using Frequency Factors

Calculating the magnitudes of extreme events by the method outlined in the above example requires that the probability distribution function be invertible, that is, given a value of T or $F(x_T) = 1 - \frac{1}{T}$, the corresponding value of x_T can be determined. Some probability distribution functions are not readily invertible, like the Normal and Pearson Type III Distributions. Thus an alternative method based on frequency factor is used for calculating the magnitudes of extreme events. Chow (1951) has shown that most frequency functions can be generalized to

$$x_T = \bar{x} + K_T s \tag{2.9}$$

where x_T is a flood of specified probability or return period T, $\bar{x}$ is the mean of the flood series, s is the standard deviation of the series; and K_T is the frequency factor and is a function of return period and type of probability distribution, as well as coefficient of skewness for skewed distributions, such as LP3.

In the event that the variable analyzed is $y = \log x$, for example as in Lognormal and LP3 Distributions, the same method is applied to the statistics for the logarithms of data using $y_T = \bar{y} + K_T s_y$, and the required value of x_T is found taking antilog of y_T.

Chow (1951) proposed the frequency factor as in Eq (2.9), and it is applicable to many probability distributions used in hydrologic frequency analysis. The K-T relationship can be expressed in mathematical terms or by a table.

Normal Distribution: From Eq (2.9) the frequency factor can be expressed as

$$K_T = \frac{x_T - \bar{x}}{s} = z \tag{2.10}$$

Thus, for Normal Distribution K_T is the same as the standard normal variable z. The value of z and hence K_T can be obtained from Table 2.2.

Lognormal Distribution: The recommended procedure for use of the Lognormal Distribution is to convert the data series to logarithms and compute:

1) $y_i = \log x_i$
2) Compute the mean, $\bar{y}$ and standard deviation s_y
3) Compute $y_T = \bar{y} + K_T s_y$

   $$K_T = \frac{y_T - \bar{y}}{s_y} = z$$

   So, K_T can be taken from Table 2.2.
4) Finally compute $x_T = anti\log y_T$

Log-Pearson Type III (LP3) Distribution: The recommended procedure for use of the LP3 Distribution is to convert the data series to logarithms and compute:

1) $y_i = \log x_i$
2) Compute the mean, $\bar{y}$ and standard deviation s_y
3) Compute coefficient of skewness

$$C_s = \frac{n\sum(y_i - \bar{y})^3}{(n-1)(n-2)s_y^{\;3}}$$

4) Compute $y_T = \bar{y} + K_T s_y$ (2.11)

where K_T is taken from Table 2.3.

5) Finally compute $x_T = anti\log y_T$

Table 2.3 gives values of the frequency factors for the LP3 Distribution for various values of return period and coefficient of skewness, C_s. When C_s =0, the frequency factor is equal to the standard normal variable z (Table 2.2).

Extreme Value I (EVI) Distribution: Chow (1951) derived the following expression for frequency factor for the EVI Distribution

$$K_T = \frac{\sqrt{6}}{\pi}\left[0.5772 + \ln\left\{\ln\left(\frac{T}{T-1}\right)\right\}\right] \tag{2.12}$$

When $x_T = \mu$, Eq (2.9) (in population term) gives $K_T = 0$ and Eq (2.12) gives T=2.33 years. This is the return period of the mean of the EVI Distribution.

Table of frequency factors for the EVI Distribution, given in Table 2.4, is taken from Haan (1977). The values computed from the above equation are equivalent to an infinite sample size in Table 2.4.

Example 2

For illustration the 5 and 50 years return period annual maximum discharges (m^3/s) for the Old Brahmaputra River near Mymensingh is calculated using the Lognormal, Log-Pearson Type III and EVI Distributions.

Year	Peak flow	y = log Q	Year	Peak flow	Log Q	Year	Peak flow	Log Q
1964	2830	3.451786	1978	2770	3.442479	1989	2180	3.338456
1965	3230	3.509202	1979	2630	3.419955	1990	2060	3.313867
1966	3490	3.542825	1980	3340	3.523746	1991	2900	3.462397
1967	3000	3.477121	1981	2690	3.429752	1992	1490	3.173186
1968	2810	3.448706	1982	2470	3.392696	1993	2060	3.313867
1969	2770	3.442479	1983	2370	3.374748	1994	1065	3.027349
1970	3250	3.511883	1984	4780	3.679427	1995	3187	3.503382
1974	3820	3.582063	1985	3070	3.487138	1996	2369	3.374565
1975	3060	3.485721	1986	1930	3.285557	1997	1973	3.295127
1976	3210	3.506505	1987	3230	3.509202	1998	3267	3.514149
1977	3550	3.550228	1988	4910	3.691081			
	Ave, $\bar{y}$ =	3.4394		Std, s_y =	0.1326		Skew, C_s =	-0.9303

Lognormal Distribution: For *T* = 50 year, *1/T* = 0.02 and $F(x) = P(Z \le z) = 0.98$ Table 2.2 is entered and *z* = 2.054 is obtained by interpolation corresponding to the tabular value of $P(Z \le z) = 0.98$. Note that the value of frequency factor can be obtained from Table 2.3 with C_s = 0.

$y_T = \bar{y} + K_T s_y$, y_{50} = 3.4394 + 2.054×0.1326 = 3.7118, $x_{50} = (10)^{3.7118} = 5150$ m^3/s

Log-Pearson Type III (LP3) Distribution: For C_s = -0.9303, the value of $K_{50} = 1.532$, so, $y_T = \bar{y} + K_T s_y$, $y_{50} = 3.4394 + 1.532 \times 0.1326 = 3.6425$, $x_{50} = (10)^{3.6425} = 4390$ m^3/s

Extreme Value I (EVI) Distribution: Eq (2.12) gives $K_{50} = 2.592$ (however, Table 2.4 gives $K_{50} = 3.007$ for n=32 years), so, $x_T = \bar{x} + K_T s$, $x_{50} = 2867.5 + 2.592 \times 804.54 = 4953$ m^3/s.

2.4.2 Graphical Frequency Analysis

The frequency of an event can be obtained by use of probability plot, which is a plot of event magnitude versus probability. As a check that a probability distribution fits a set of hydrologic data, the data are plotted on specially designed probability paper that linearizes the distribution function. The plotted data are then fitted with a straight line for interpolation and extrapolation purposes. Determining the probability to assign a data point is commonly referred to as determining probability position.

Plotting Positions: Plotting position refers to the probability value assigned to each piece of data to be plotted. If *n* is the total number of values to be plotted and *m* is the rank of a value in a list ordered by descending magnitude, the exceedance probability of the m^{th} largest value x_m is, for large *n*,

$$P(X \geq x_m) = \frac{m}{n}$$

However this simple formula (known as California's formula) produces a probability of 100%, which implies that the largest sample value is the largest possible value. A value of 100% cannot be plotted on many probability paper (Haan 1977). To overcome this limitation other formulas have been proposed. Several plotting position formulas are given below.

Plotting position formulas

California	(m/n)
Hazen	(m-0.5)/n
Beard	1 - $(0.5)^{1/n}$
Weibull	m/(n+1)
Gringorten	(m-0.44)/(n+0.12)
Chegodayev	(m-0.3)/(n+0.4)
Blom	(m-3/8)/(n+1/4)
Tukey	(3m-1/3n+1)

The technique in all cases is to arrange the data in increasing or decreasing order of magnitude and to assign order number *m* to the ranked values. The most efficient formula for computing plotting positions for unspecified distribution and the one now commonly used for most sample data, is

$$P = \frac{m}{n+1}$$

When *m* is ranked from lowest to highest, *P* is an estimate of the probability of values being equal to or less than the ranked value, that is, *P(X≥x)*; when the rank is from highest to lowest, *P* is *P(X≤x)*.

Example 3

As an example, probability plotting analysis of the annual maximum discharges (m^3/s) of the Old Brahmaputra near Mymensingh is performed. Also plotted data are compared with best-fit EVI Distribution.

Rank	Peak flow	Plotting position*	Rank	Peak flow	Plotting position	Rank	Peak flow	Plotting position
m	*Q*	*P*	*m*	*Q*	*P*	*m*	*Q*	*P*
1	4910	0.017	12	3187	0.3604	23	2470	0.702
2	4780	0.049	13	3070	0.391	24	2370	0.733
3	3820	0.0797	14	3060	0.422	25	2369	0.765
4	3550	0.111	15	3000	0.453	26	2180	0.7958
5	3490	0.142	16	2900	0.484	27	2060	0.827
6	3340	0.173	17	2830	0.5156	28	2060	0.858
7	3267	0.204	18	2810	0.547	29	1973	0.889
8	3250	0.235	19	2770	0.578	30	1930	0.920
9	3230	0.267	20	2770	0.610	31	1490	0.951
10	3230	0.298	21	2690	0.640	32	1065	0.983
11	3210	0.329	22	2630	0.671			

Sample size n = 32
Ave = 2867.531
Std dev = 804.5443, Skew = 0.37198
* Gringorten, P = (m−0.44)/(n+1−0.88)

First the data are ranked from largest (m=1), as shown below to smallest (m=n=32). Gringorten's plotting formula (b=0.44) was used since data are being fitted to EVI Distribution. For example, for m=1, the exceedance probability ($Q \geq 4910$ m^3/s) = (m−0.44)/(n+1−0.88) = (1−0.44)/(32+0.12) = 0.56/32.12 = 0.017. Similarly all the plotting positions are calculated and plotted on EVI paper (Fig 2.8). The plotted points represent the empirical distribution obtained using 32 observed peak flows.

Several points on the best-fit EVI line are calculated using Eq (2.9) as follows:

T=5 years,	$P(Q \geq q) = 0.20$	$K_5 = 0.719$,	$Q_5 = 3446$ m^3/s
T=25 years,	$P(Q \geq q) = 0.04$	$K_{25} = 2.044$,	$Q_{25} = 4511$ m^3/s
T=50 years,	$P(Q \geq q) = 0.02$	$K_{50} = 2.592$,	$Q_{50} = 4952$ m^3/s
T=100 years,	$P(Q \geq q) = 0.01$	$K_{100} = 3.137$,	$Q_{100} = 5390$ m^3/s

A straight line is drawn through the calculated points to obtain the best-fit EVI Distribution line. In this example the plotted points show good-fit with EVI Distribution.

2.5 Goodness-of-fit Tests

The goodness of fit of a probability distribution can be tested by comparing the theoretical and sample values of the relative frequency or the cumulative frequency function. In the case of the relative frequency function, the χ^2 – test is used and with cumulative frequency function the Kolmogorov-Smirnov test is used.

Chi-Square Test: The test statistic is given by

$$\chi^2 = \sum_{i=1}^{k} \frac{n\left[f_s(x_i) - p(x_i)\right]^2}{p(x_i)} \qquad (2.13)$$

where k is the number of intervals; the sample value of the relative frequency of interval i is, $f_s(x_i) = n_i/n$; the theoretical value of the relative frequency function (also called incremental probability function) is $p(x_i) = F(x_i) - F(x_{i-1})$. It may be noted that $nf_s(x_i) = n_i$, the observed number of occurrences in interval i, and $np(x_i)$ is the corresponding expected number of occurrences in interval i.

To describe the χ^2 test, the χ^2 probability distribution must be defined. A χ^2 distribution with $\upsilon = k\text{-}l\text{-}1$ degrees of freedom (l is the number of parameters used in fitting the proposed distribution) is the distribution for the sum of squares of υ independent standard normal random variables z_i. The critical χ^2 distribution function is tabulated (in Table 2.5) from Haan (1977). A confidence level is chosen for the test; it is often expressed as $1\text{-}\alpha$, where α is termed the significance level.

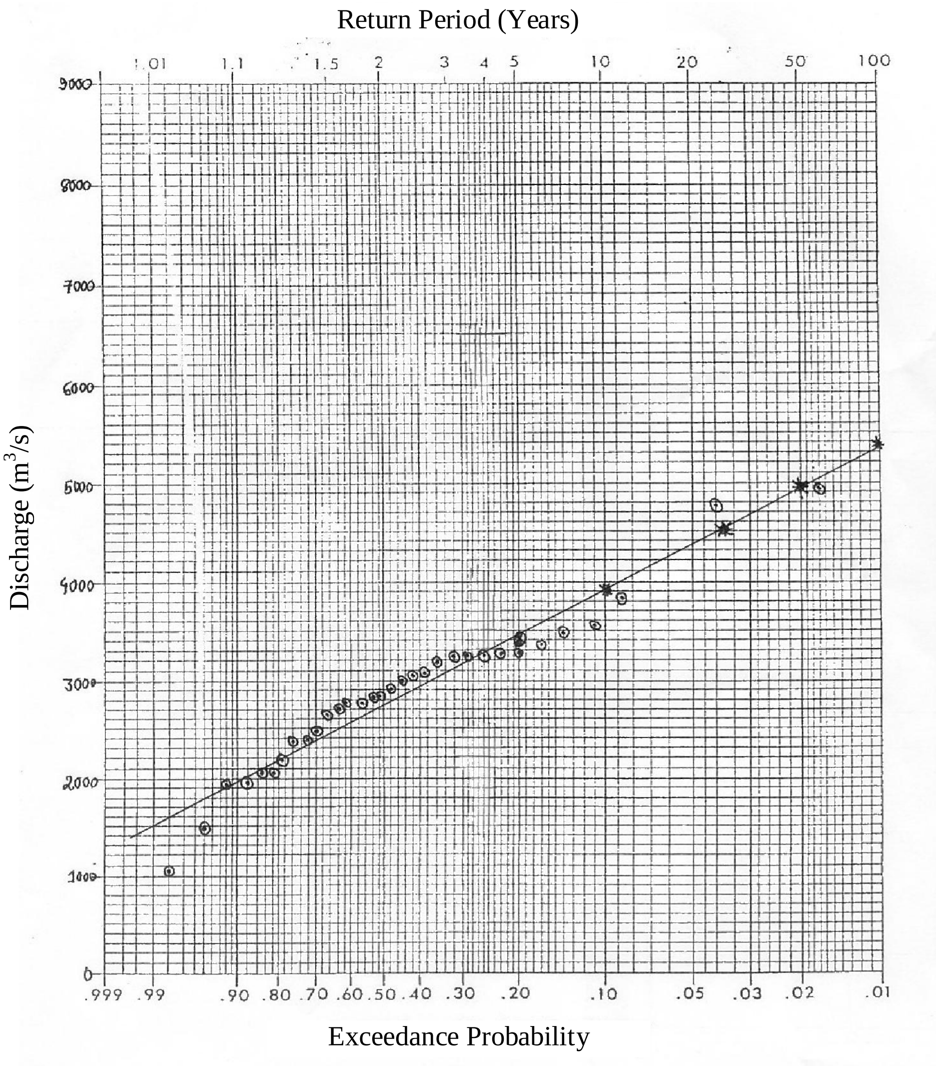


Fig 2.8 EVI probability plot for annual peak flows of Old Brahmaputra, Mymensingh

Example 4

Chi square test is used to determine whether EVI Distribution adequately fits the Old Brahmaputra river annual peak flow data.

Thirty two peak flow observation are divided into six class intervals. The number or frequency of observations, n_i in each class is counted. The observed or sample values of relative frequency $f_s(x_i)$ is calculated with $n = 32$. For example, for the second class interval $f_s(x_2) = 8/32 = 0.25$. The observed cumulative frequency found by summing up the relative frequencies.

To fit EVI Distribution, the parameters α and u are calculated as before (α = 627.62, u = 2505.27, $\bar{x}$ = 2867.5 and s = 804.54 m^3/s). The theoretical cumulative frequencies corresponding to the upper limit of each of class interval is calculated by finding reduced variate y from Eq (2.5) and the $F(x)$ by Eq (2.4). For example, for the second class interval

$$y_2 = \frac{(x_2 - u)}{\alpha} \quad \text{and} \quad F(x_2) = e^{-e^{-y_2}} = 0.36479$$

$p(x_2) = P(1750 \le X \le 2500) = F(2500) - F(1750) = 0.32904$

The value of 0.32904 is entered under the expected relative frequency corresponding to the class interval 1750-2500 in the table below.

The calculation is repeated for other class intervals and summed to obtain χ^2 = 2.35. This is the computed χ^2 value.

To test the goodness of fit, this is compared with the critical χ^2 value to be obtained from tabular values as shown below.

Class limit		Num of obs.	Obs frequency	Obs cum frequency	Reduced variate	Expected cum frequency	Expected relative frequency	Chi square
Lower limit	Upper limit	n_i	$f_s(x_i)$	$F_s(x_i)$	y_i	$F(x_i)$	$p(x_i)$	χ^2
1000	1750	2	0.0625	0.0625	-1.2033	0.03575	0.03575	0.64050
1750	2500	8	0.25	0.3125	-0.0084	0.36479	0.32904	0.60757
2500	3250	14	0.4375	0.75	1.1866	0.73693	0.37214	0.36734
3250	4000	6	0.1875	0.9375	2.3815	0.91174	0.17481	0.02948
4000	4750	1	0.03125	0.96875	3.5765	0.97242	0.06068	0.45676
4750	5500	1	0.03125	1.0	4.7716	0.99157	0.01915	0.24465
Total		32	1.00		Computed Chi square χ^2			2.3463

For a confidence level of 90%, from Table 2.5, the critical Chi square for $\upsilon = k\text{-}l\text{-}1$ = 6-2-1 = 3 degree of freedom, χ^2 = 6.25. Since the computed Chi square value of 2.35 is less than the critical value of 6.25, the data fits EVI Distribution adequately.

Kolmogorov-Smirnov Test: The theoretical and sample values of the cumulative frequency are compared with the Kolmogorov-Smirnov (S-K) test. The test statistic D, which is based on deviations of the sample distribution function $P(x)$ from the completely specified continuous hypothetical distribution function $P_o(x)$, such that:

$$D = \max\left|P(x) - P_o(x)\right|$$

Developed by Kolmogorov (Kite 1988) in 1933, the test requires that the value of D computed from the sample distribution be less than the tabulated value of D (Table 2.6) at the required confidence level. Hoque et al. (1998) reported that Kolmogorov-Smirnov test for Gumbel's Extremal Distribution gives better result in Bangladesh

2.6 Testing for Outliers

Outliers are data points that depart significantly from the trend of the remaining data. The retention or deletion of these outliers can significantly affect the magnitude of statistical parameters computed from the data, especially for small samples. Procedures for treating outliers require judgement involving both mathematical and hydrologic considerations. According to the Water Resources Council (1981), if the station skew is greater than +0.4, tests for high outliers are considered first; if the station skew is less than -0.4, tests for low outliers are considered first. Where the station skew is between ±0.4, tests for both high and low outliers should be applied before eliminating any outliers form the data set.

The following frequency equation can be used to detect high outliers:

$$y_H = \bar{y} + K_n s_y \tag{2.14}$$

where y_H is the high outlier threshold in log units and K_n (used in one-sided tests that detect outliers at the 10% level of significance in normally distributed data) is as given in Table 2.7 for sample size n. If the logarithms of the values in a sample are greater than y_H in the above equation, then they are considered high outliers. Flood peaks considered high outliers should be compared with historic flood data flood information at nearby sites. Historic flood data comprise information on unusually extreme events outside of the systematic record. According to the Water Resources Council (1981), if information is available that indicates a high outlier is the maximum over an extended period of time, the outlier is treated as historic flood data and excluded from analysis. If useful historic information is not available to compare to high outliers, then the outliers should be retained as part of the systematic record.

A similar equation can be used to detect low outliers:

$$y_L = \bar{y} - K_n s_y \tag{2.15}$$

where y_L is the low outlier threshold in log units. Flood peaks considered low outliers are detected from the record and a conditional probability adjustment can be applied.

Example 6

Using the data for the Old Brahmaputra, determine if there are any high or low outliers for the sample.

Given: $n = 32$, $\bar{y} = 3.4394, s_y = 0.1326, C_s = -0.9303$; Table 2.7 gives $K_n = 2.591$.

$y_H = \bar{y} + K_n s_y$ =3.4394+2.591×(0.1326) = 3.783; then $Q_H = 10^{3.783} =$ 6067 m^3/s.

The largest recorded value (4910 m^3/s) does not exceed the threshold value, so there are no high outliers in this sample.

Similarly, $y_L = \bar{y} - K_n s_y =$ 3.4394-2.591×(0.1326) = 3.0958; then $Q_L = 10^{3.0958} =$ 1247 m^3/s. The 1994 peak flow of 1065 m^3/s is less than Q_L and so is considered a low outlier.

2.7 Reliability of Analysis

The reliability of results of frequency analysis depends on how well the assumed probabilistic model applies to a given set of hydrologic data. A measurement of variability in x_T is the standard error of estimate.

Standard Error: The standard error of estimate s_T is a measure of the standard deviation of event magnitudes computed from samples about the true event magnitude. The standard error of estimate can be expressed as (Kite 1977):

$$s_T = \delta_T \frac{s}{\sqrt{n}} \tag{2.16}$$

where s is the standard deviation of the original sample of size n. The quantity δ_T depends on the return period and can be expressed in terms of frequency factor as follows:

Normal Distribution: $\delta_T = \sqrt{\left(2 + K_T^2\right)}$, recall for normal distribution K_T is the same as the standard normal variate z for an exceedance probability $1/T$.

EVI Distribution: $\delta_T = \left[1 + 1.39K_T + 1.1K_T^2\right]^{1/2}$

LP3 Distribution: The values of δ_T for different coefficient of skewness and return period are given in Table 2.8.

Confidence Limits: Statistical estimates are often presented with a range, or confidence interval, within which the true value can be reasonably be expected to lie. The size of the confidence interval depends on the confidence level β. The upper boundary and lower boundary values of the confidence interval are called the confidence limits. Corresponding to the confidence level β there is a significance level α, given by $\alpha = (1- \beta)/2$.

For example if $\beta =$ 90%, then α = (1-0.9)/2 = 0.05 or 5%.

Confidence limits can be plotted on flood frequency curves to indicate reliability of discharge estimate for various recurrence intervals. An approximate confidence limit or confidence interval for x_T has the upper and lower bounds as given below.

$$x_T \pm s_T z_\alpha = x_T \pm \left(\delta_T \frac{s}{\sqrt{n}}\right) z_\alpha \qquad (2.17)$$

where z_α is the value of the standard normal variable with exceedance probability α =0.05 or cumulative probability 0.95 (Table 2.2). The confidence limits can also be determined for normally distributed data using the non-central t Distribution given in Table 2.9 (Kendall and Stuart 1967).

Example 7

As illustration the standard error of estimate and the 90% confidence limits on the EVI probability plot for the Old Brahmaputra River is computed below.

From the example given above, the estimated flood magnitudes for various return periods, T using EVI Distribution and corresponding K_T are taken to estimate standard error and confidence limits:

For the given sample, n = 32, $\bar{x} = 2867.531$ m^3/s, s = 804.5443 m^3/s, C_s = 0.37198. When T=5 years, Eq (2.12) gives K_T= 0.719, x_T = 3446 m^3/s.

$$s_T = \left[1 + 1.39 \times 0.719 + 1.1 \times 0.719^2\right]^{\frac{1}{2}} \frac{804.5443}{\sqrt{32}} = 227.9$$

The 90% confidence limits with $z_\alpha = 1.645$ for α =0.05 are:

$x_T \pm s_T z_\alpha = 3446 \pm 227.9 \times 1.645 =$ [3821 and 3071]

Similarly for other x_T values the lower and upper confidence limits are:

T=1.1 year,	x_T = 1957,	K_T = -1.132,	s_T =130.0,	C. limits [1702, 2212]
T =2 year,	x_T = 2735,	K_T = -0.1642,	s_T =127.3,	C. limits [2526, 2944]
T =2.33 year,	x_T = 2868,	K_T = 0.0,	s_T =142.2,	C. limits [2627, 3110]
T =5 year,	x_T = 3446,	K_T = 0.719,	s_T =227.9,	C. limits [3071, 3821]
T =25 year,	x_T = 4511,	K_T = 2.043,	s_T =400.26,	C. limits [3853, 5170]
T =50 year,	x_T = 4952,	K_T = 2.591,	s_T =478.98,	C. limits [4164, 5740]
T =100 year,	x_T = 5390,	K_T = 3.315,	s_T =557.92,	C. limits [4472, 6308]

With these calculated lower and upper confidence limits the reliability band is drawn on the EVI best fit line in Fig 2.9.

2.8 Transfer of Discharge and Water Level

Correlation and Interpolation

Often data are not available directly at a site and some means of transposing or extending the data are necessary. Several methods of transposing and extending station data are mentioned in Neill (1973). These include interpolation of flood frequency values, regional frequency analysis and correlation of floods between two stations. A technique of transferring high water levels up- or down-stream, or interpolating between two stations is outlined here briefly. Such procedure is affected greatly by changes in channel cross-section and slope, and by presence of major tributaries. Nevertheless, in some cases a reasonably consistent picture may be obtained if observed maximum stages at various points are plotted along a river-slope profile.

Hydrodynamic Models

Now a days versatile hydrodynamic simulation models are available and can be used to predict discharge and water level at points where measured data on discharge and water level are not available Using available discharge and water level at suitably located up- and down-stream stations on the same river or in a homogeneous region, discharge and water level can be simulated at other points within the model reach or region.

2.9 Bankful Discharge

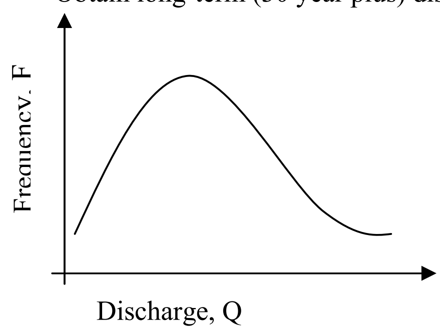
The bankful discharge of a river may be defined as the discharge which is contained within the banks of the river. This is the state of maximum velocity in the channel, and therefore of maximum competence for the transport of sediment load.

Bankful discharge is assumed to be a major determinant of the size and shape of a river channel, but it is difficult to measure in the field, and a wide variety of field procedures exist for this measurement. Quoting return periods for bankful discharge is a tricky business because over a dozen methods are available, but the frequency of its occurrence seems to vary with climatic regimes.

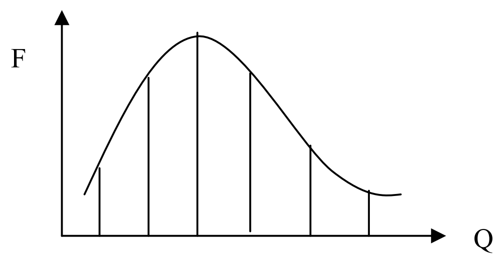
2.10 Dominant Discharge Analysis

The dominant discharge is the flow doing most geomorphic work and it is, therefore, the channel forming discharge. It probably does not correspond to bankful flow on any river. The dominant or channel forming flow represents an alternative benchmark criterion to bankful flow when analyzing channel form and process. To estimate the dominant discharge the following steps are followed:

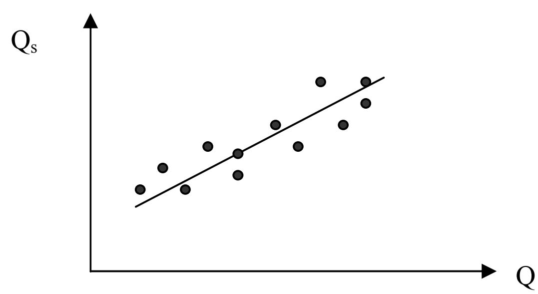
1. Obtain long-term (30 year plus) distribution of flows for gauging station.



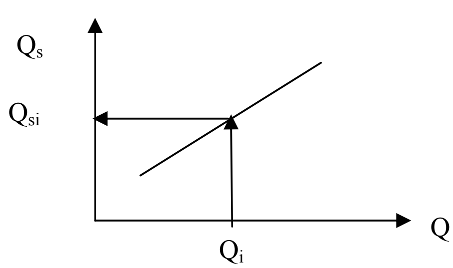
2. Split this into discrete of equal class interval. For Brahmaputra, let us try initially 5,000 m^3/s class interval, and check sensitivity of results to this choice of interval. Find mid point of each class.



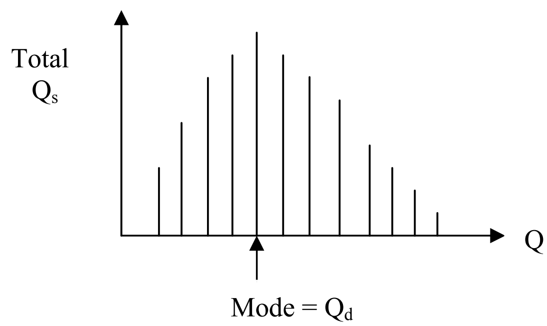
3. Obtain the most reliable sediment rating curve for the gauging station. Ideally this should be for total load, but a suspended load curve may be used provided that suspended load makes up most of the total load, as is usually the case.



4. Use the sediment rating curve to find the sediment transport rate (tons/sec) for the mid-point discharge of each flow class.



5. Multiply the sediment transport rate for each discharge class with the frequency of that class to obtain the total sediment load transported by that flow during the period; plot this as a histogram.



6. From the histogram, identify the mode. This corresponds to the dominant discharge. Determine the magnitude of Qd = dominant discharge and use the flow duration curve to establish its return period.

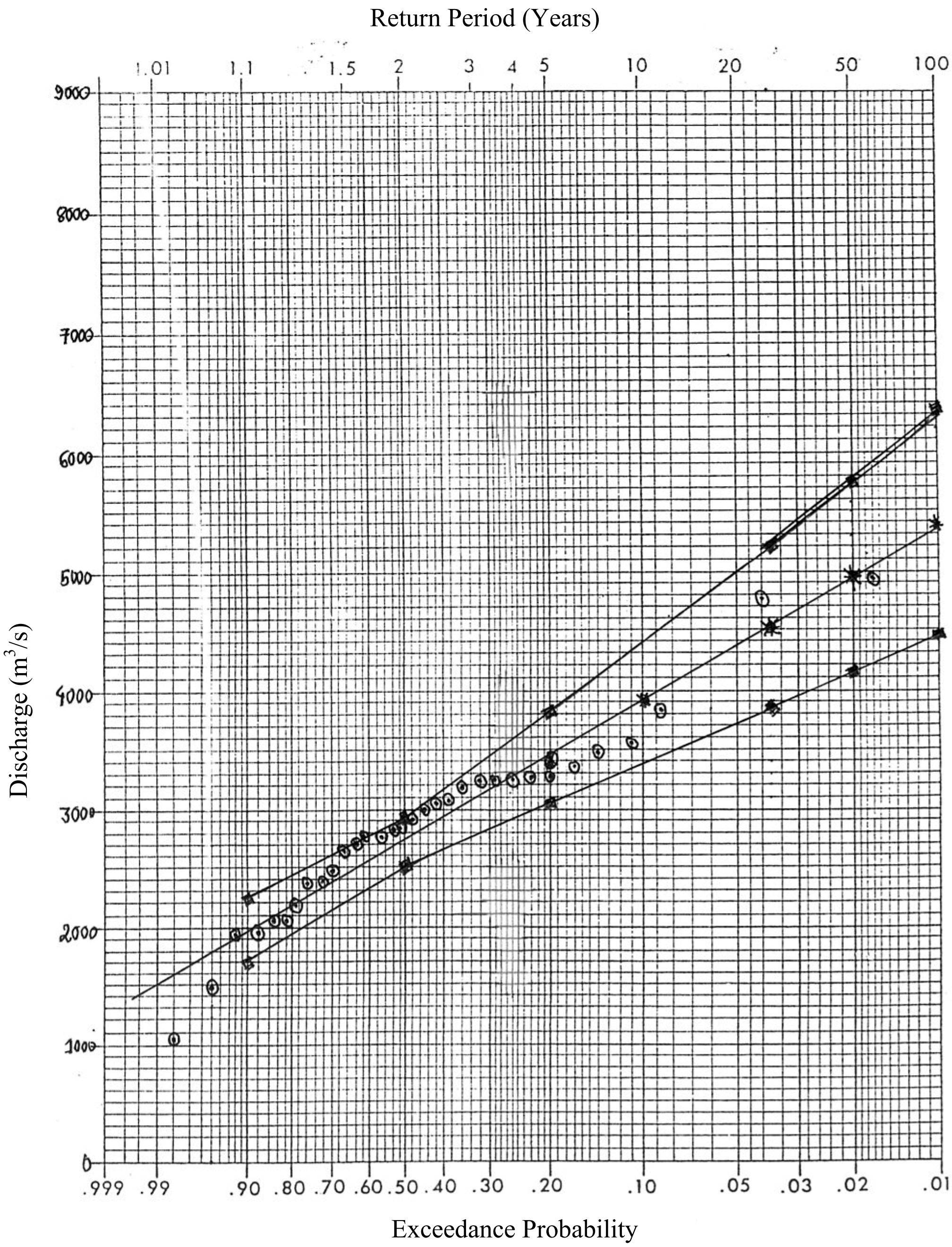


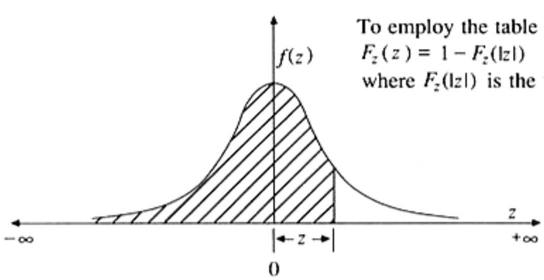
Fig 2.9 EVI probability plot and 90% confidence limits for annual peak flows of Old Brahmaputra, Mymensingh

Table 2.2 Cumulative probability of the Standard Normal Distribution

Cumulative probability of the standard normal distribution

z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936
2.5	0.9938	0.9940	0.9941	0.9943	0.9945	0.9946	0.9948	0.9949	0.9951	0.9952
2.6	0.9953	0.9955	0.9956	0.9957	0.9959	0.9960	0.9961	0.9962	0.9963	0.9964
2.7	0.9965	0.9966	0.9967	0.9968	0.9969	0.9970	0.9971	0.9972	0.9973	0.9974
2.8	0.9974	0.9975	0.9976	0.9977	0.9977	0.9978	0.9979	0.9979	0.9980	0.9981
2.9	0.9981	0.9982	0.9982	0.9983	0.9984	0.9984	0.9985	0.9985	0.9986	0.9986
3.0	0.9987	0.9987	0.9987	0.9988	0.9988	0.9989	0.9989	0.9989	0.9990	0.9990
3.1	0.9990	0.9991	0.9991	0.9991	0.9992	0.9992	0.9992	0.9992	0.9993	0.9993
3.2	0.9993	0.9993	0.9994	0.9994	0.9994	0.9994	0.9994	0.9995	0.9995	0.9995
3.3	0.9995	0.9995	0.9995	0.9996	0.9996	0.9996	0.9996	0.9996	0.9996	0.9997
3.4	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9997	0.9998

Source: Grant, E. L., and R. S. Leavenworth, *Statistical Quality and Control*, Table A, p.643, McGraw-Hill, New York, 1972. Used with permission.



To employ the table for $z < 0$, use
$F_z(z) = 1 - F_z(|z|)$
where $F_z(|z|)$ is the tabulated value.

Table 2.3 Frequency factors for Pearson Type III Distribution

K_T values for Pearson Type III distribution (positive skew)

Skew coefficient C_s or C_w	Return period in years: 2	5	10	25	50	100	200
	Exceedence probability: 0.50	0.20	0.10	0.04	0.02	0.01	0.005
3.0	−0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	−0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	−0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	−0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	−0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	−0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	−0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	−0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	−0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	−0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	−0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	−0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	−0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	−0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	−0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	−0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	−0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	−0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	−0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	−0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	−0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	−0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	−0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	−0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	−0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	−0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	−0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	−0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.2	−0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	−0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0	0.842	1.282	1.751	2.054	2.326	2.576

Table 2.3 Continued

K_T values for Pearson Type III distribution (negative skew)

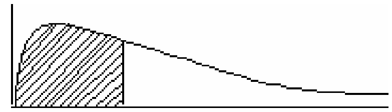
Skew coefficient C_s or C_w	Return period in years: 2	5	10	25	50	100	200
	Exceedence probability: 0.50	0.20	0.10	0.04	0.02	0.01	0.005
–0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
–0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388
–0.3	0.050	0.853	1.245	1.643	1.890	2.104	2.294
–0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
–0.5	0.083	0.856	1.216	1.567	1.777	1.955	2.108
–0.6	0.099	0.857	1.200	1.528	1.720	1.880	2.016
–0.7	0.116	0.857	1.183	1.488	1.663	1.806	1.926
–0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
–0.9	0.148	0.854	1.147	1.407	1.549	1.660	1.749
–1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
–1.1	0.180	0.848	1.107	1.324	1.435	1.518	1.581
–1.2	0.195	0.844	1.086	1.282	1.379	1.449	1.501
–1.3	0.210	0.838	1.064	1.240	1.324	1.383	1.424
–1.4	0.225	0.832	1.041	1.198	1.270	1.318	1.351
–1.5	0.240	0.825	1.018	1.157	1.217	1.256	1.282
–1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
–1.7	0.268	0.808	0.970	1.075	1.116	1.140	1.155
–1.8	0.282	0.799	0.945	1.035	1.069	1.087	1.097
–1.9	0.294	0.788	0.920	0.996	1.023	1.037	1.044
–2.0	0.307	0.777	0.895	0.959	0.980	0.990	0.995
–2.1	0.319	0.765	0.869	0.923	0.939	0.946	0.949
–2.2	0.330	0.752	0.844	0.888	0.900	0.905	0.907
–2.3	0.341	0.739	0.819	0.855	0.864	0.867	0.869
–2.4	0.351	0.725	0.795	0.823	0.830	0.832	0.833
–2.5	0.360	0.711	0.771	0.793	0.798	0.799	0.800
–2.6	0.368	0.696	0.747	0.764	0.768	0.769	0.769
–2.7	0.376	0.681	0.724	0.738	0.740	0.740	0.741
–2.8	0.384	0.666	0.702	0.712	0.714	0.714	0.714
–2.9	0.390	0.651	0.681	0.683	0.689	0.690	0.690
–3.0	0.396	0.636	0.666	0.666	0.666	0.667	0.667

Source: U. S. Water Resources Council (1981).

Table 2.4 Frequency factors for Extreme Value I Distribution

	Return Period								
Sample	5	10	15	20	25	50	75	100	1000
15	0.967	1.703	2.117	2.410	2.632	3.321	3.721	4.005	6.265
20	0.919	1.625	2.023	2.302	2.517	3.179	3.563	3.836	6.006
25	0.888	1.575	1.963	2.235	2.444	3.088	3.463	3.729	5.842
30	0.866	1.541	1.922	2.188	2.393	3.026	3.393	3.653	5.727
35	0.851	1.516	1.891	2.152	2.354	2.979	3.341	3.598	
40	0.838	1.495	1.866	2.126	2.326	2.943	3.301	3.554	5.576
45	0.829	1.478	1.847	2.104	2.303	2.913	3.268	3.520	
50	0.820	1.466	1.831	2.086	2.283	2.889	3.241	3.491	5.478
55	0.813	1.455	1.818	2.071	2.267	2.869	3.219	3.467	
60	0.807	1.446	1.806	2.059	2.253	2.852	3.200	3.446	
65	0.801	1.437	1.796	2.048	2.241	2.837	3.183	3.429	
70	0.797	1.430	1.788	2.038	2.230	2.824	3.169	3.413	5.359
75	0.972	1.423	1.780	2.029	2.220	2.812	3.155	3.400	
80	0.788	1.417	1.773	2.020	2.212	2.802	3.145	3.387	
85	0.785	1.413	1.767	2.013	2.205	2.793	3.135	3.376	
90	0.782	1.409	1.762	2.007	2.198	2.785	3.125	3.367	
95	0.780	1.405	1.757	2.002	2.193	2.777	3.116	3.357	
100	0.779	1.401	1.752	1.998	2.187	2.770	3.109	3.349	5.261
α	0.719	1.305	1.635	1.866	2.044	2.592	2.911	3.137	4.936

Table 2.5 χ^2 Distribution



$X^2_{a,\,v}$

DOF ν	$x^2_{.995}$	$x^2_{.99}$	$x^2_{.975}$	$x^2_{.95}$	$x^2_{.90}$	$x^2_{.75}$	$x^2_{.50}$	$x^2_{.25}$	$x^2_{.10}$	$x^2_{.05}$	$x^2_{.025}$	$x^2_{.01}$	$x^2_{.005}$
1	7.88	6.63	5.02	3.84	2.71	1.32	0.455	0.102	0.0158	0.0039	0.0010	0.0002	0.0000
2	10.6	9.21	7.38	5.99	4.61	2.77	1.39	0.575	.211	.103	.0506	.0201	.0100
3	12.8	11.3	9.35	7.81	6.25	4.11	2.37	1.21	.584	.352	.216	.115	.072
4	14.9	13.3	11.1	9.49	7.78	5.39	3.36	1.92	1.06	.711	.484	.297	.207
5	16.7	15.1	12.8	11.1	9.24	6.63	4.35	2.67	1.61	1.15	.831	.554	.412
6	18.5	16.8	14.4	12.6	10.6	7.84	5.35	3.45	2.20	1.64	1.24	.872	.676
7	20.3	18.5	16.0	14.1	12.0	9.04	6.35	4.25	2.83	2.17	1.69	1.24	.989
8	22.0	20.1	17.5	15.5	13.4	10.2	7.34	5.07	3.49	2.73	2.18	1.65	1.34
9	23.6	21.7	19.0	16.9	14.7	11.4	8.34	5.90	4.17	3.33	2.70	2.09	1.73
10	25.2	23.2	20.5	18.3	16.0	12.5	9.34	6.74	4.87	3.94	3.25	2.56	2.16
11	26.8	24.7	21.9	19.7	17.3	13.7	10.3	7.58	5.58	4.57	3.82	3.05	2.60
12	28.3	26.2	23.3	21.0	18.5	14.8	11.3	8.44	6.30	5.23	4.40	3.57	3.07
13	29.8	27.7	24.7	22.4	19.8	16.0	12.3	9.30	7.04	5.89	5.01	4.11	3.57
14	31.3	29.1	26.1	23.7	21.1	17.1	13.3	10.2	7.79	6.57	5.63	4.66	4.07
15	32.8	30.6	27.5	25.0	22.3	18.2	14.3	11.0	8.55	7.26	6.26	5.23	4.60
16	34.3	32.0	28.8	26.3	23.5	19.4	15.3	11.9	9.31	7.96	6.91	5.81	5.14
17	35.7	33.4	30.2	27.6	24.8	20.5	16.3	12.8	10.1	8.67	7.56	6.41	5.70
18	37.2	34.8	31.5	28.9	26.0	21.6	17.3	13.7	10.9	9.39	8.23	7.01	6.26
19	38.6	36.2	32.9	30.1	27.2	22.7	18.3	14.6	11.7	10.1	8.91	7.63	6.84
20	40.0	37.6	34.2	31.4	28.4	23.8	19.3	15.5	12.4	10.9	9.59	8.26	7.43
21	41.4	38.9	35.5	32.7	29.6	24.9	20.3	16.3	13.2	11.6	10.3	8.90	8.03
22	42.8	40.3	36.8	33.9	30.8	26.0	21.3	17.2	14.0	12.3	11.0	9.54	8.64
23	44.2	41.6	38.1	35.2	32.0	27.1	22.3	18.1	14.8	13.1	11.7	10.2	9.26
24	45.6	43.0	39.4	36.4	33.2	28.2	23.3	19.0	15.7	13.8	12.4	10.9	9.89
25	46.9	44.3	40.6	37.7	34.4	29.3	24.3	19.9	16.5	14.6	13.1	11.5	10.5
26	48.3	45.6	41.9	38.9	35.6	30.4	25.3	20.8	17.3	15.4	13.8	12.2	11.2
27	49.6	47.0	43.2	40.1	36.7	31.5	26.3	21.7	18.1	16.2	14.6	12.9	11.8
28	51.0	48.3	44.5	41.3	37.9	32.6	27.3	22.7	18.9	16.9	15.3	13.6	12.5
29	52.3	49.6	45.7	42.6	39.1	33.7	28.3	23.6	19.8	17.7	16.0	14.3	13.1
30	53.7	50.9	47.0	43.8	40.3	34.8	29.3	24.5	20.6	18.5	16.8	15.0	13.8
40	66.8	63.7	59.3	55.8	51.8	45.6	39.3	33.7	29.1	26.5	24.4	22.2	20.7
50	79.5	76.2	71.4	67.5	63.2	56.3	49.3	42.9	37.7	34.8	32.4	29.7	28.0
60	92.0	88.4	83.3	79.1	74.4	67.0	59.3	52.3	46.5	43.2	40.5	37.5	35.5
70	104.2	100.4	95.0	90.5	85.5	77.6	69.3	61.7	55.3	51.7	48.8	45.4	43.3
80	116.3	112.3	106.6	101.9	96.6	88.1	79.3	71.1	64.3	60.4	57.2	53.5	51.2
90	128.3	124.1	118.1	113.1	107.6	98.6	89.3	80.6	73.3	69.1	65.6	61.8	59.2
100	140.2	135.8	129.6	124.3	118.5	109.1	99.3	90.1	82.4	77.9	74.2	70.1	67.3

Source: Catherine M. Thompson, Table of percentage points of the χ^2 distribution, Biometrika, Vol. 32 (1941), by permission of the author and publisher.

Table 2.6 Kolmogorov-Smirnov Distribution

Sample size (n)	Significance Level				
	.20	0.15	0.10	0.05	0.01
1	.900	.925	.950	.975	.995
2	.684	.726	.776	.842	.929
3	.565	.597	.642	.708	.829
4	.494	.725	.564	.624	.734
5	.446	.474	.510	.563	.669
6	.410	.436	.470	.521	.618
7	.381	.405	.438	.486	.577
8	.358	.381	.411	.457	.543
9	.339	.360	.388	.432	.514
10	.322	.342	.368	.409	.486
11	.307	.326	.352	.391	.468
12	.295	.313	.338	.375	.450
13	.284	.302	.325	.361	.433
14	.274	.292	.314	.349	.418
15	.266	.283	.304	.338	.404
16	.258	.274	.295	.328	.391
17	.250	.266	.286	.318	.380
18	.244	.259	.278	.309	.370
19	.237	.252	.272	.301	.361
20	.231	.246	.264	.294	.352
25	.21	.22	.24	.264	.32
30	.19	.20	.22	.242	.29
35	.18	.19	.21	.23	.27
40				.21	.25
50				.19	.23
60				.17	.21
70				.16	.19
80				.15	.18
90				.14	
100				.14	
Asymptotic Formula	$\frac{1.70}{\sqrt{n}}$	$\frac{1.14}{\sqrt{n}}$	$\frac{1.22}{\sqrt{n}}$	$\frac{1.36}{\sqrt{n}}$	$\frac{1.63}{\sqrt{n}}$

Source: Journal American Statistical Association 47:425-441, 1952.Z.W. Birnbaum.

Table 2.7 Outlier test K_n values

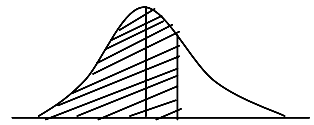
Sample size n	K_n	Sample size n	K_n	Sample size n	K_n	Sample size n	K_n
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.700	80	2.940
15	2.247	29	2.549	43	2.710	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3.000
18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.760	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129
23	2.448	37	2.650	55	2.804	-	-

Source: U. S. Water Resources Council, 1981. This table contains one sided 10-percent significance level K_n values for the normal distribution.

Table 2.8 δ_T values for LP3 Distribution

	Return period								
γ	1.053	1.111	1.25	2	5	10	20	50	100
0.2	1.507	1.253	1.107	1.083	1.231	1.499	1.881	2.498	3.017
0.3	1.429	1.194	1.075	1.087	1.261	1.561	1.985	2.665	3.236
0.4	1.36	1.137	1.042	1.092	1.291	1.623	2.092	2.842	3.471
0.5	1.302	1.082	1.008	1.099	1.32	1.685	2.2	3.027	3.722
0.6	1.257	1.032	0.974	1.107	1.35	1.745	2.31	3.22	3.987
0.7	1.228	0.986	0.94	1.118	1.38	1.805	2.421	3.42	4.265
0.8	1.217	0.947	0.904	1.13	1.41	1.863	2.532	3.626	4.555
0.9	1.226	0.917	0.868	1.145	1.44	1.921	2.643	3.837	4.856
1.	1.256	0.899	0.832	1.161	1.472	1.976	2.753	4.052	5.167
1.1	1.307	0.894	0.795	1.18	1.506	2.03	2.862	4.27	5.488
1.2	1.378	0.905	0.759	1.2	1.542	2.082	2.969	4.49	5.815
1.3	1.467	0.933	0.724	1.222	1.58	2.133	3.072	4.711	6.149
1.4	1.571	0.98	0.693	1.246	1.622	2.182	3.173	4.933	6.489
1.5	1.688	1.043	0.666	1.27	1.669	2.231	3.271	5.153	6.832
1.6	1.815	1.123	0.646	1.296	1.721	2.279	3.364	5.371	7.177
1.7	1.949	1.218	0.637	1.321	1.779	2.327	3.453	5.585	7.523
1.8	2.088	1.325	0.641	1.346	1.843	2.376	3.537	5.796	7.868
1.9	2.227	1.442	0.661	1.37	1.915	2.426	3.617	6.	8.211
2.	2.364	1.568	0.699	1.393	1.995	2.48	3.692	6.198	8.55
2.1	2.496	1.699	0.754	1.414	2.083	2.538	3.762	6.388	8.883
2.2	2.619	1.834	0.826	1.431	2.179	2.602	3.829	6.57	9.209
2.3	2.73	1.97	0.912	1.446	2.285	2.673	3.893	6.742	9.526
2.4	2.824	2.104	1.012	1.456	2.4	2.753	3.954	6.904	9.834
2.5	2.898	2.235	1.123	1.461	2.523	2.843	4.015	7.055	10.129

Table 2.9 t Distribution



$t_{a,\, y}$

r	$t_{.995}$	$t_{.99}$	$t_{.975}$	$t_{.93}$	$t_{.90}$	$t_{.80}$	$t_{.75}$	$t_{.70}$	$t_{.60}$	$t_{.55}$
1	63.66	31.82	12.71	6.31	3.08	1.376	1.000	0.727	0.325	0.158
2	9.92	6.96	4.30	2.92	1.89	1.061	0.816	0.617	0.289	0.142
3	5.84	4.54	3.18	2.35	1.64	0.978	0.765	0.584	0.277	0.137
4	4.60	3.75	2.78	2.13	1.53	0.941	0.741	0.569	0.271	0.134
5	4.03	3.36	2.57	2.02	1.48	0.920	0.727	0.559	0.267	0.132
6	3.71	3.14	2.45	1.94	1.44	0.906	0.718	0.553	0.265	0.131
7	3.50	3.00	2.36	1.90	1.42	0.896	0.711	0.549	0.263	0.130
8	3.36	2.90	2.31	1.86	1.40	0.889	0.706	0.546	0.262	0.130
9	3.25	2.82	2.26	1.83	1.38	0.883	0.703	0.543	0.261	0.129
10	3.17	2.76	2.23	1.81	1.37	0.879	0.700	0.542	0.260	0.129
11	3.11	2.72	2.20	1.80	1.36	0.876	0.697	0.540	0.260	0.129
12	3.06	2.68	2.18	1.78	1.36	0.873	0.695	0.539	0.259	0.128
13	3.01	2.65	2.16	1.77	1.35	0.870	0.694	0.538	0.259	0.128
14	2.98	2.62	2.14	1.76	1.34	0.868	0.692	0.537	0.258	0.128
15	2.95	2.60	2.13	1.75	1.34	0.866	0.691	0.536	0.258	0.128
16	2.92	2.58	2.12	1.75	1.34	0.865	0.690	0.535	0.258	0.128
17	2.90	2.57	2.11	1.74	1.33	0.863	0.689	0.534	0.257	0.128
18	2.88	2.55	2.10	1.73	1.33	0.862	0.688	0.534	0.257	0.127
19	2.86	2.54	2.09	1.73	1.33	0.861	0.688	0.533	0.257	0.127
20	2.84	2.53	2.09	1.72	1.32	0.860	0.687	0.533	0.257	0.127
21	2.83	2.52	2.08	1.72	1.32	0.859	0.686	0.532	0.257	0.127
22	2.82	2.51	2.07	1.72	1.32	0.858	0.686	0.532	0.256	0.127
23	2.81	2.50	2.07	1.71	1.32	0.858	0.685	0.532	0.256	0.127
24	2.80	2.49	2.06	1.71	1.32	0.857	0.685	0.531	0.266	0.127
25	2.79	2.48	2.06	1.71	1.32	0.856	0.684	0.531	0.256	0.127
26	2.78	2.48	2.06	1.71	1.32	0.856	0.684	0.531	0.256	0.127
27	2.77	2.47	2.05	1.70	1.31	0.855	0.684	0.531	0.256	0.127
28	2.76	2.47	2.05	1.70	1.31	0.855	0.683	0.530	0.256	0.127
29	2.76	2.46	2.04	1.70	1.31	0.854	0.683	0.530	0.256	0.127
30	2.75	2.46	2.04	1.70	1.31	0.854	0.683	0.530	0.256	0.127
40	2.70	2.42	2.02	1.68	1.30	0.851	0.681	0.529	0.255	0.126
50	2.66	2.39	2.00	1.67	1.30	0.848	0.679	0.527	0.254	0.126
120	2.62	2.36	1.98	1.66	1.29	0.845	0.677	0.526	0.254	0.126
α	2.58	2.33	1.96	1.645	1.28	0.842	0.674	0.524	0.253	0.126

CHAPTER THREE

Bank Erosion and General Protection Measures

3.1 Introduction

Main causes and mechanisms of bank erosion followed by the possible protection measures with their application ranges are discussed in this chapter.

3.2 River Instability and Bank Erosion

The factors controlling stability of a river include the amount and rate of supply of water and sediment into the system, catchment geology and vegetation pattern (type and extent) in the catchment. As the factors change over time, the river responds by altering shape, form and position.

In large, wide and heavily sediment-laden rivers of Bangladesh, the deeper channel or thalweg and anabranching or secondary channels frequently relocate their position within the river corridor. In a braided river, high rate of sedimentation and subsequent formation of channel bars or chars also instigate oblique flows to attack bankline. Curved bank or bends are created in this way which speeds up local erosion rate. Protection of a part of an unstable river reach may also induce similar local instability in downstream reaches.

3.3 Mechanism and Processes of Bank Erosion

Bank erosion processes are mainly two types: (i) direct fluid entrainment and (ii) mass failure. The former is the detachment of grains from the bank surface, followed by fluvial entrainment. The latter is the collapse of a part of the bank en-masse, in response to geotechnical instability.

3.3.1 Surface Erosion of Banks

The main impacts responsible for surface erosion at river banks are (Fig. 3.1):

i) Current induced shear stress
ii) Wave loads (wind-generated waves; ship and boat-generated waves)
iii) Seepage (excessive pore pressure)
iv) Surface runoff
v) Mechanical action (desiccation, ship impact, activities of humans and animals)

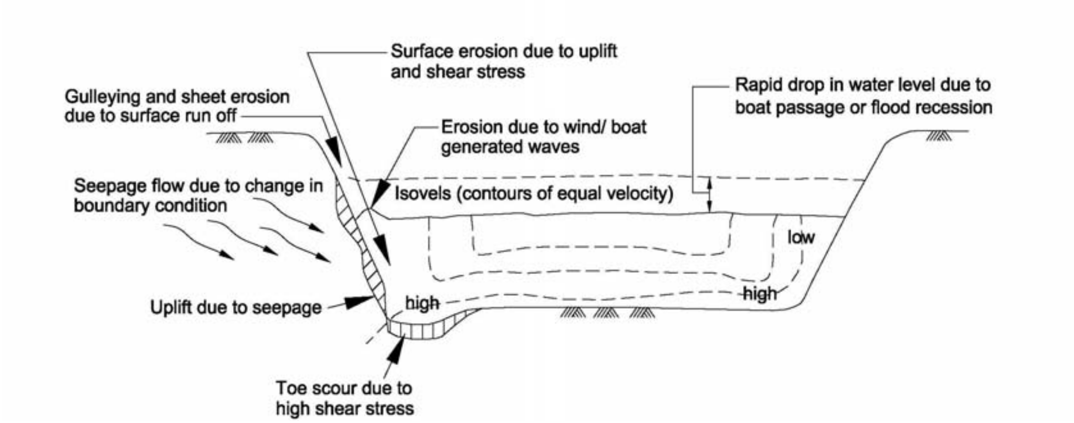


Fig. 3.1 Processes of surface erosion (adopted Hamphill and Bramley, 1989).

The main factor of bank erosion is the shear stress induced by current flow. A typical shear stress distribution in a trapezoidal cross-section of a straight channel is shown in Fig. 3.2.

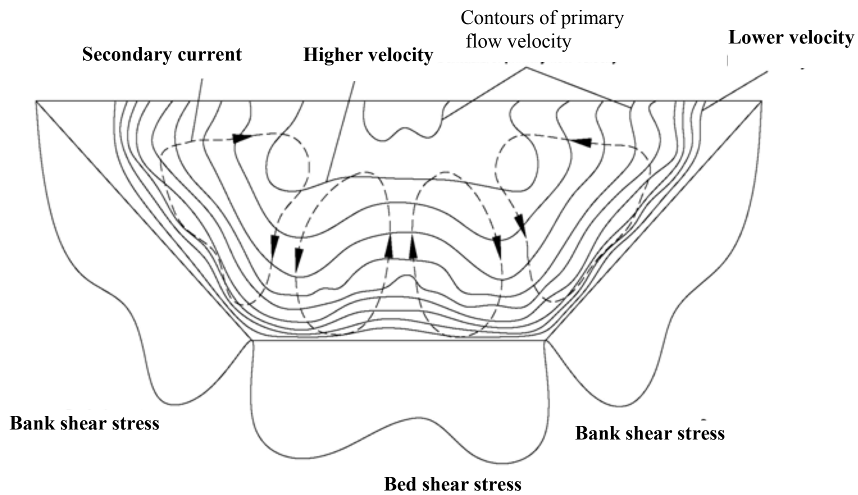


Fig. 3.2 Contours of primary and secondary flows as well as the shear stress distribution in a trapezoidal channel (Hemphill and Bramley, 1989)

The secondary currents generated by inertia forces, are the key factor for the asymmetric cross sectional profiles. Secondary flows also exist in straight river sections due to the velocity gradient in the primary currents (Fig. 3.2). Significant variations of the shear stress and velocity along straight and meandering sections is shown in Fig. 3.3.

Obstructions to the flow as well as variations of the roughness of the river bank 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. Scouring occurs at the outer bank of a meander bend during high flows. The eroded material is transported as bed load and suspended load which are deposited in areas of lower 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.

The influence of wave action is important along the bank line above low water level i.e., above the transition between main channel and adjacent floodplain of a river. In this zone the wave energy is transformed to wave run-up and wave run-down, increasing shear stress and erosion. In addition, depending on the breaker type (wave breaking phenomenon) extremely high pressure heads may occur especially under plunging breakers. These cause considerable destruction hitting the bank slope.

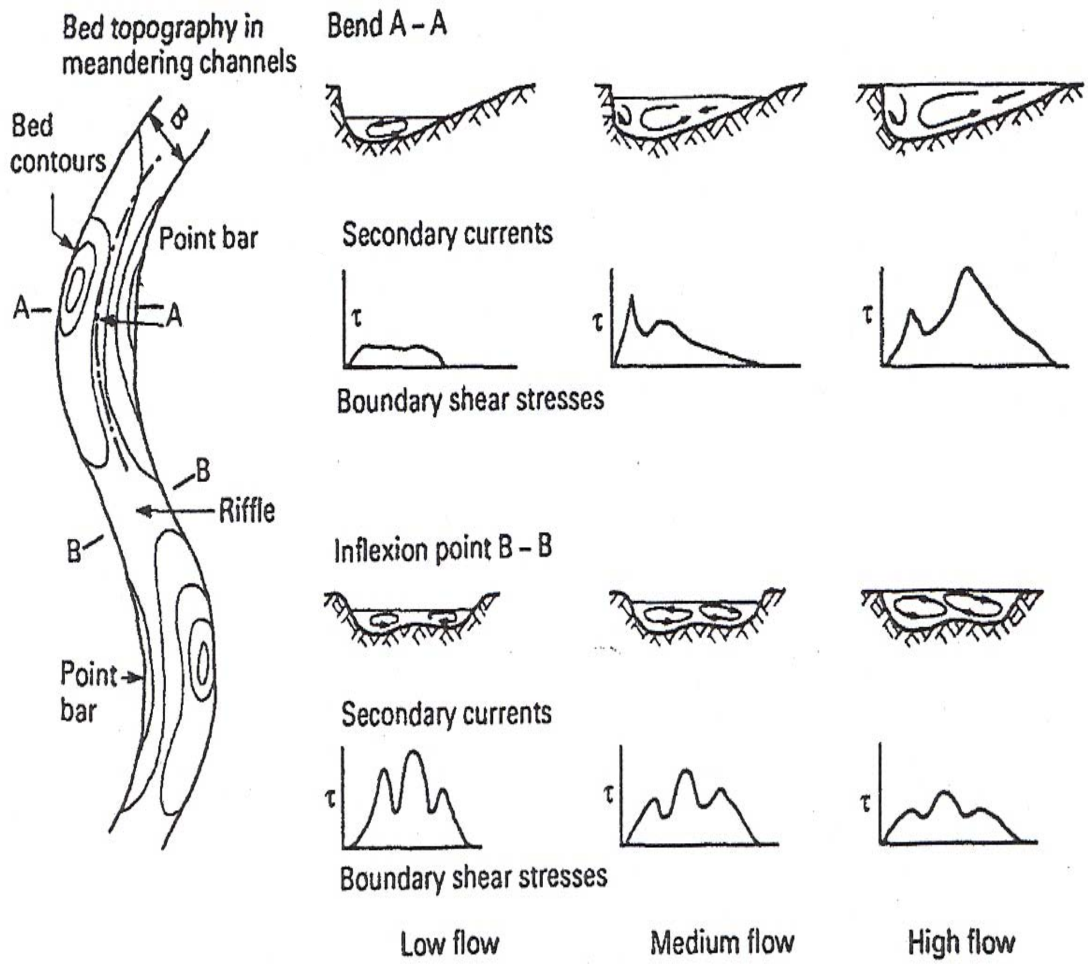


Fig 3.3 Secondary currents and boundary shear stress in a meandering channel (after Hey, 1986, in Hamphill and Bramley, 1989)

3.3.2 Mass Failure of Banks

Mass failure of river banks are of three types (i) *slip failures and,* (ii) *block failures*, illustrated in Fig. 3.4 and Fig 3.5. The actual failing of a river bank may not follow immediately after an impact and may take several days. On the other hand, a failure may occur suddenly when active surface erosion and toe scouring is prevalent or an additional (surcharge) load is applied to the bank. The risk of mass failure is increased during heavy rain and during quick fall of river stages after flood. Typical bank failure of major rivers in Bangladesh is shown in Fig. 3.5.

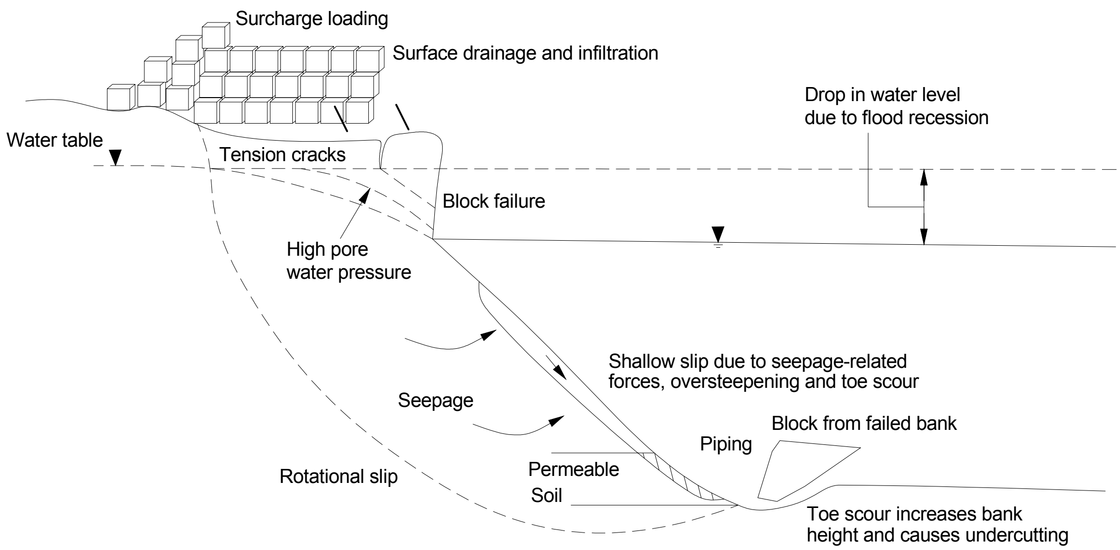


Fig 3.4 Processes responsible for mass failure of a river bank

Fig 3.5a Typical mass failure of banks

Fig 3.5b Typical mass failure of banks

The different failure modes of slip and block failures (Hey and Tovey 1989) are given below:

Slip Failures

i) In case of non-cohesive material and a low bank angle the failure surface is usually approximately parallel to the slope angle (Fig. 3.6a). Seepage can substantially reduce the stability of the bank, whereas vegetation will normally help to stabilize against failure.

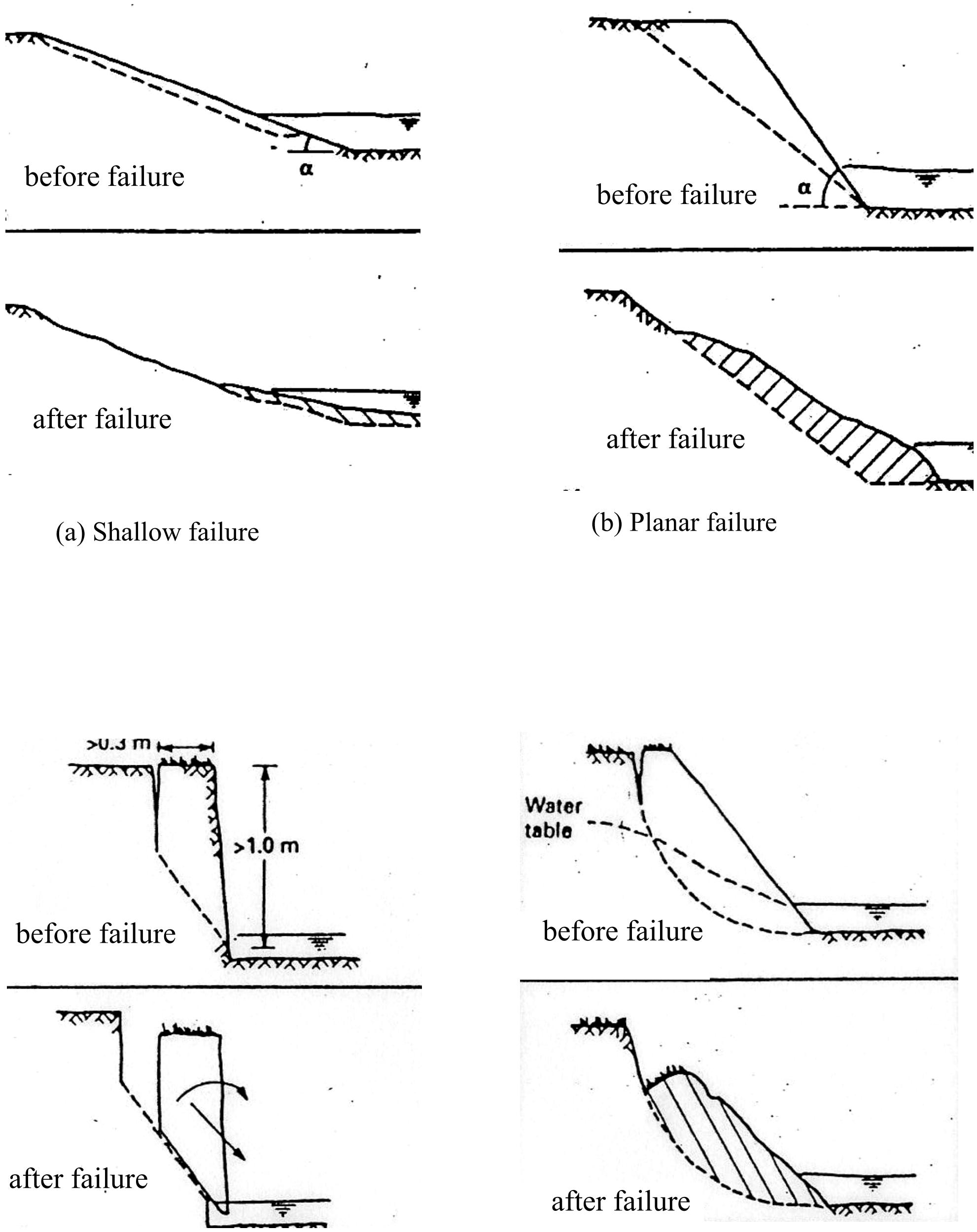
ii) Steep or almost vertical banks of non-cohesive material can fail along a plane or slightly curved surface (Fig. 3.6b). This is often the case when the river water level is low relative to the total bank height.

iii) If relatively deep tension cracks have developed on the surface of the river bank, failure occurs by sliding and/or toppling (Fig. 3.6c). This failure mode is little affected by the groundwater table, but is more likely if the crack fills with water.

iv) 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 (Fig. 3.6d).

v) 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 section (Fig. 3.6e). 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.

vi) If the outside bank of an eroding meander bends 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.



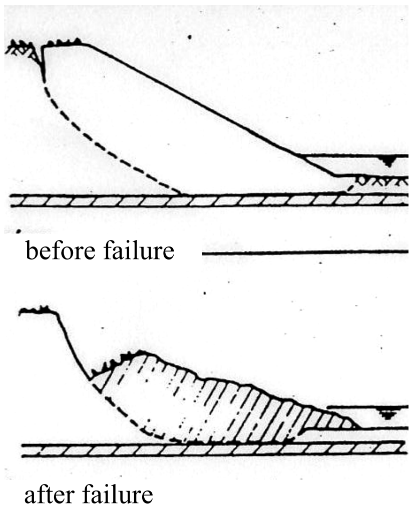
(a) Shallow failure

(b) Planar failure

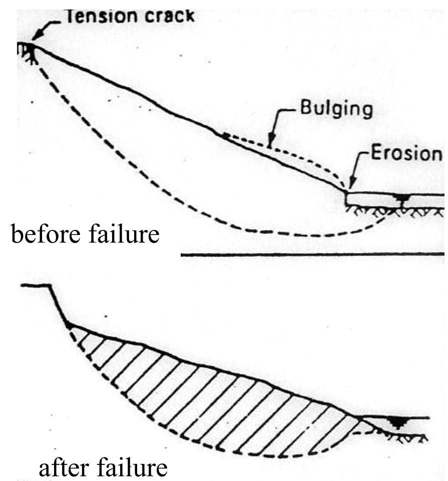
(c) Planar failure with tension crack

(d) Rotational failure (homogeneous material)

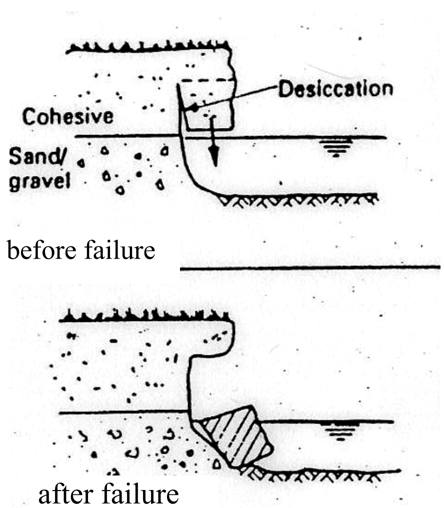
Fig. 3.6 Different modes of river bank failure: immediately before and after failure (Hey and Tovey, 1989)



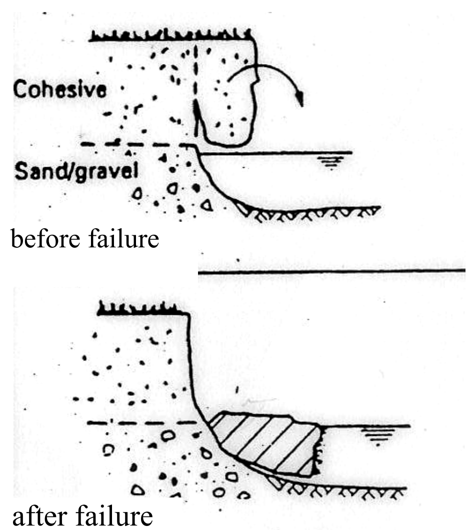
(e) Failure with
weak zone



(f) Deep-seated rotational failure
(landslides)



(g) Failure of composite
bank in tension



(h) Failure of composite
bank in shear

Fig. 3.6 (continued) Different modes of river bank failure: immediately before and after failure (Hey and Tovey, 1989)

In many river banks, upper part of the bank slope consists of finer material (e.g., silt and clay) while the lower part contains sand and other coarser grains. In that case an under-cut may develop in the lower part of the composite bank which is more frequently exposed to flowing water. The upper part then fails under tension or shear and falls on the toe area as a complete block (Fig. 3.6g).

3.4 Assessment of Erosion Potential

Monitoring and observations provide the required information for assessing erosion potential. Assessment can be based on historic information, or field survey of existing site conditions. Available aerial photographs, old maps, surveying notes, design files and river survey data, along with gauging station records and interviews of local people can provide information of any past and recent channel movement or instabilities.

In Bangladesh, satellite images from different sources (SPOT, LANDSAT, IRS etc.) are used. The images are used to delineate banklines and calculate yearly rates of bankline shifting.

3.5 General Methods of Bank Protection

3.5.1 General Concept

In general, three relevant concepts of erosion counter measures are in existence:

(i) River training measures, which are instream structures that are intended to influence the flow conditions or channel properties downstream of the location of the measure (active measures), e.g., bottom vanes, floating screens bandalling.

(ii) Structures, which are constructed as a protrusion from the river bank to decrease the hydraulic impacts in front of the area to be protected (partly active or partly passive measures), e.g., groynes and spur.

(iii) Structures to protect the bankline directly without active interference with the water flow to reduce its erosive strength before attacking the bank slope (passive measures), e.g., revetments.

It should always be kept in mind that whatever bank protection measure is carried out, the succeeding river response will certainly call for compensatory measures in the following years. Therefore, adaptation and supplementary measures must always be seen as an inseparable element of the protection measures.

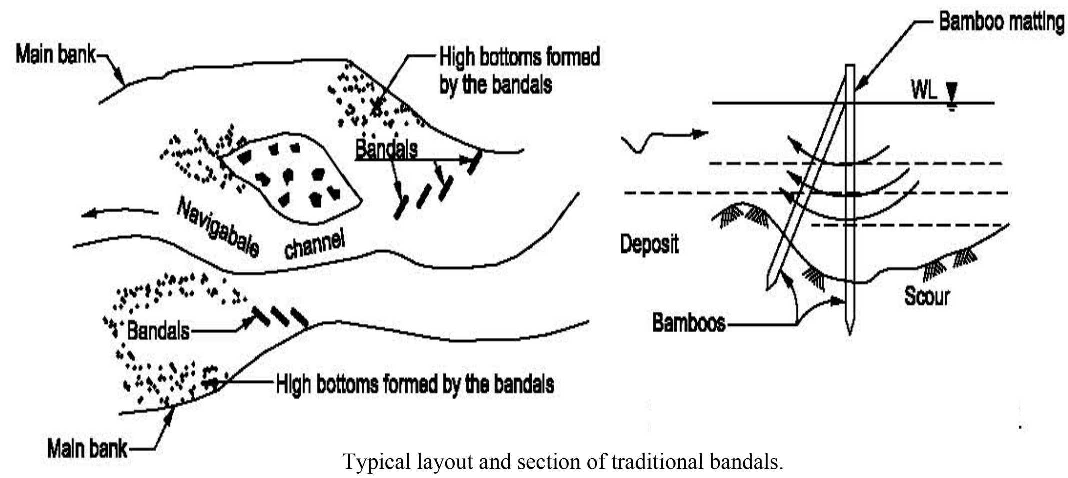
3.5.2 Traditional and Low Cost Measures

A large variety of traditional and low cost measures is existent, such as (a) *bandals*, (b) *floating screens*, (c) *intelligent dredging*, (d) *closure dams*, (e) *artificial cut-offs*, (f) *porcupines* and (g) *retards.* In general, these measures are restricted to a certain range of applications and comparatively low hydraulic impacts. A brief details on some of the methods are given below:

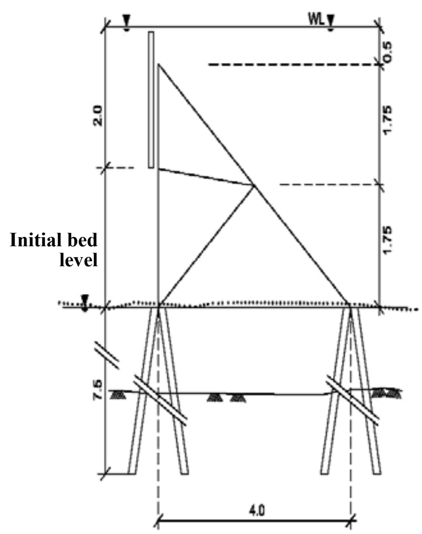
(a) Bandals

Typical layout of bandals are shown in Fig 3.7. Bandals consist of a framework of bamboo driven into the riverbed during low water stage and supported by struts. Bamboo mats are fixed to the framework near the water level. Bandal structures are commonly applied to improve or maintain the depth of river channels for 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 along the thalweg. 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).

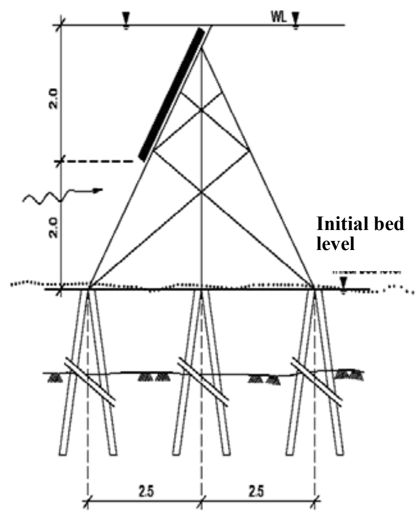
Due to the fact that blockage ratio of bandals change with water depth, their efficiency depends strongly on the actual water depth. Since the stage and depth of flow vary with time, a good performance is achieved only during particular time period when an optimal blockage ratio of the individual bandals is achieved. These structures may be used for water depth up to 4m to 5m.



Typical layout and section of traditional bandals.



Improved low-flow bandal



Improved high-flow bandal

Fig. 3.7 Bandals for channel improvements: traditional (above) and improved design (below)

(b) Floating Screens

These consist of a floating element e.g. a pontoon or barge to which a vertical screen is fixed (Fig. 3.8). The screens are adjustable in depth to maintain optimal blockage.

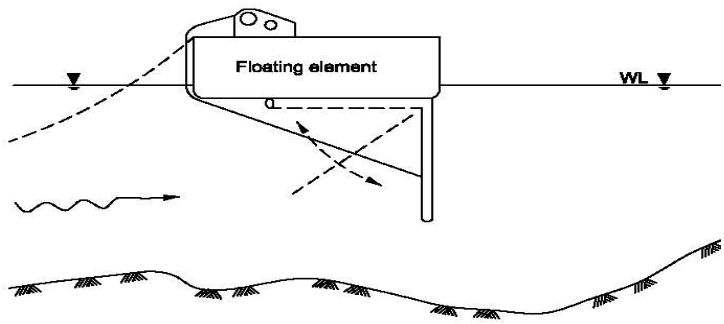


Fig. 3.8 Floating screens for channel improvements

(c) Dredging Measures

This is 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 (Fig. 3.9). Artificial cut-offs and closure dams are specific types of dredging measures.

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

(d) Closure Dams

Secondary channels in a braided river or river bends may be closed by earthen dams, which are built during low water stages. Typical locations are at bifurcations or at narrow areas of a channel (Fig.3.9).

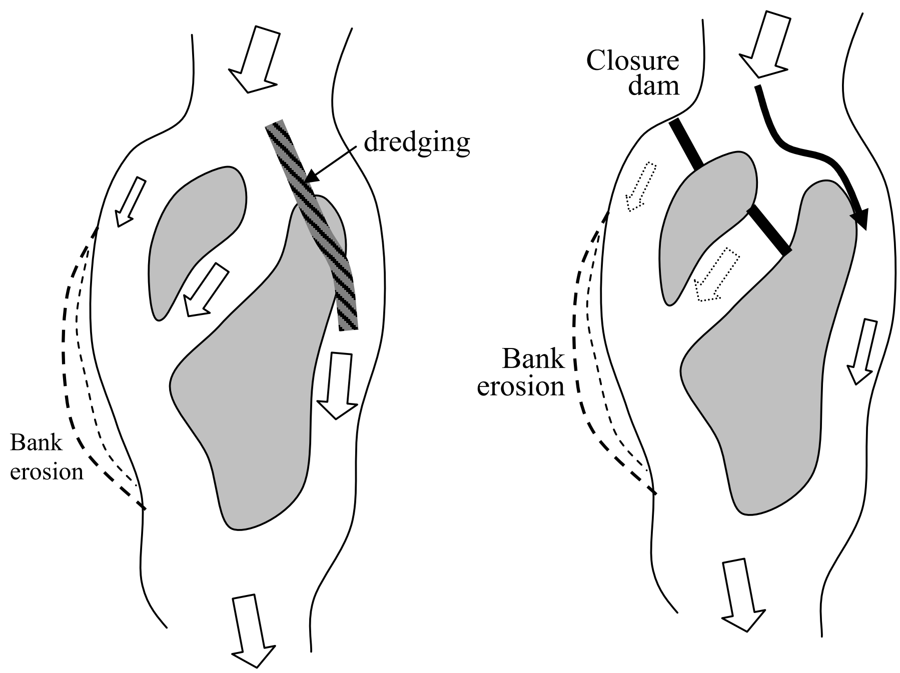


Fig. 3.9 Dredging measures and closure dams for erosion control

(e) Artificial Cut-off

Cut-offs may be used to reduce the erosion attack at well developed concave bends (Fig. 3.10) by excavating a channel in full dimensions or as a pilot channel. This channel is eroded to the final extent during rising water stages, provided that the existing sediment is fairly mobile sandy soil. The rate of flow directed to the pilot channel should be at least 25% to 30% of the bankfull discharge. Cut-off results in an increase in water slope, bed slope and flow velocity, which usually lead to local erosion especially at downstream.

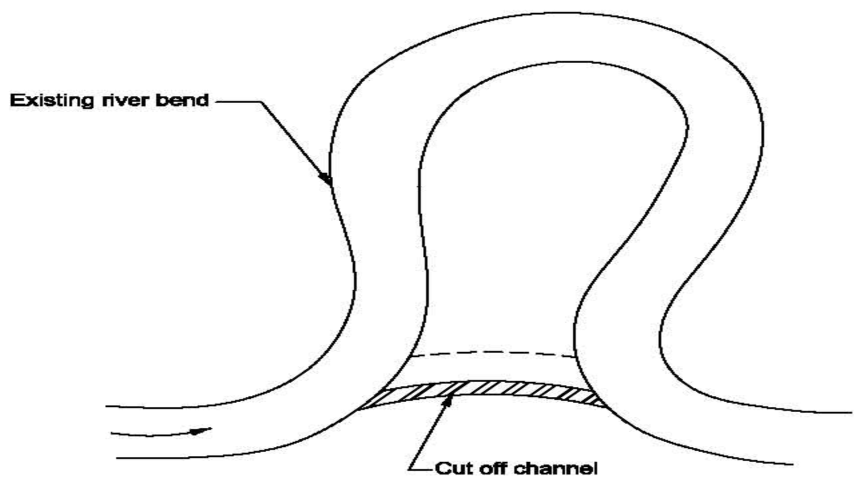


Fig. 3.10 Artificial cutoff of river bend

(f) Porcupine

These are box-like bamboo frames with leg extensions of the bamboo units from all corners (Fig. 3.11). Porcupines are ballasted by filling the box with brickbats and sunk or placed in predetermined positions. Porcupines are deployed in staggered arrangements in rows and columns to cover the underwater surface of slope and toe of the river bank which has to be protected from erosion.

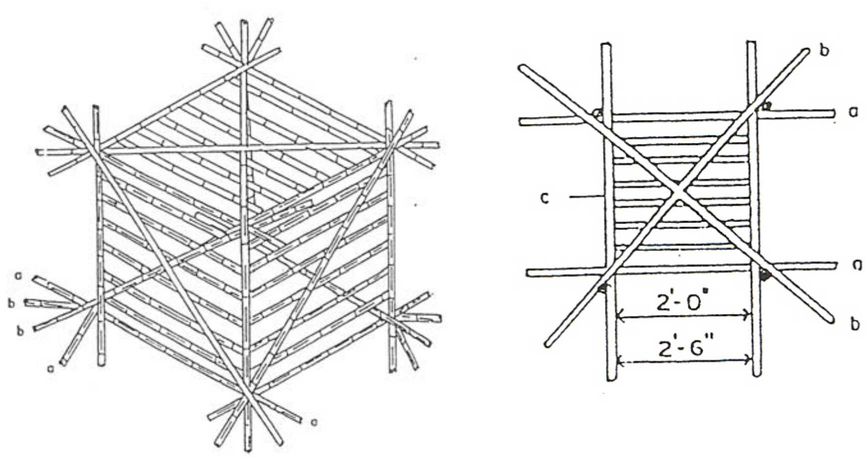


Fig. 3.11 Isometric and plan view of porcupine

Porcupines are effective in sediment laden tidal rivers where velocity of flow is moderate. These are not suitable for river with deep scour depths, high flow velocity, high turbulence and coarser sediment load. For sustained effectiveness, usually, additional porcupines are needed to dump in the protected site during the subsequent years after first deployment.

(g) Retards

Retards also termed as pallasiding are structures longitudinal to the flow and constructed in the river in front of the bank to be protected (Fig. 3.12). Under favourable conditions, the space between the retards and the bank gets silted up. The structures can be made submersible too.

Retards may be made by:

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

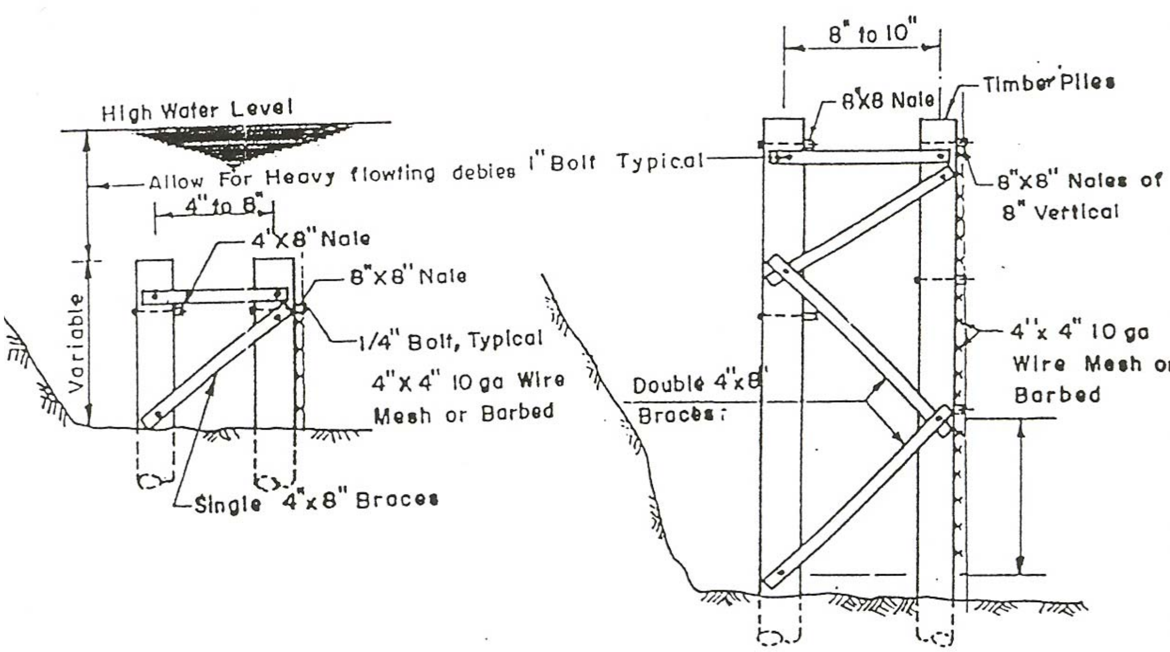


Fig. 3.12 Retards constructed near the bank

(h) Bottom Vanes

Bottom vanes are small foil type flow training structures mounted vertically on the riverbed with an angle to the prevailing flow. *Iowa vanes* and *submerged vanes* are also similar measures. Although introduced long time ago, this type of vanes are now well known due to a number of publications during the last three decades related to laboratory experiments and field applications of vanes by Odgaard and his colleagues working in the Iowa Institute of Hydraulic Research (IIHR).

Vanes can be constructed with steel piles as well as bamboo, tree trunks (bullah) etc. A view of a bottom vane constructed with low cost materials installed in the Elanjani River is shown in Fig. 3.13 (Hossain et al., 2006).

Several arrays of vanes are normally installed to create a vane field in the bend of the river. The location and orientation of the vanes are of paramount importance to obtain desired result. To encounter undesired local scour during floods and subsequent failure of the vane itself, sand-cement bags are used around the base of the vane. The combined effect is that appreciable deposition of sediment occurs at the outer bank where usually erosion takes place without vane condition.

Fig. 3.13 A bottom vane constructed with indigenous materials

3.5.3 Biotechnical River Bank Protection

Biotechnical river bank protection utilizes living plant materials to reinforce soil and to stabilize slopes. Plants can be used as the primary structural component or in combination with inert materials like rock, concrete, and steel. Plants, however, provides satisfactory protection on shallow, low gradient banks, but are unsuitable where steep banks prevail.

Grasses (Vetiver, Napier) can be effective to sheet and rill erosion. For protecting the river bank, Woody species are commonly used for slope stabilization. Durba grass is preferably used for the land-sided slope of main embankment or for the upper area of river-sided slopes.

Sonneratia and Avicenea are considered to be suitable for newly accreted land in tidal and saline condition. Where applicable, afforestation of the foreshore areas (mangrove forest or non-mangrove e.g., Jhau forest) is useful to reduce erosion due to wave action.

3.5.4 Standard Practices

In this category two types of structures known as groynes and revetments are used in Bangladesh. These are explained below.

3.5.4.1 Groynes

Groynes are stone, gravel, rock, earth or pile structures built at an angle to the river bank to deflect flowing water away from the bank. The words *spur* and *groyne* are used almost synonymously. However, the term spur is more frequently used in Bangladesh when the structure is shorter with respect to the channel width.

A typical groyne has mainly two parts: *head*, and *shank*. The part joining the head and the bank/embankment is the shank. Maximum flow convergence and divergence occur around the groyne head and deep scour holes are formed. So additional protection measures around the head of the groyne is required. The upstream and downstream sides of the shank also need protection but with less amount of erosion resisting materials than the groyne head.

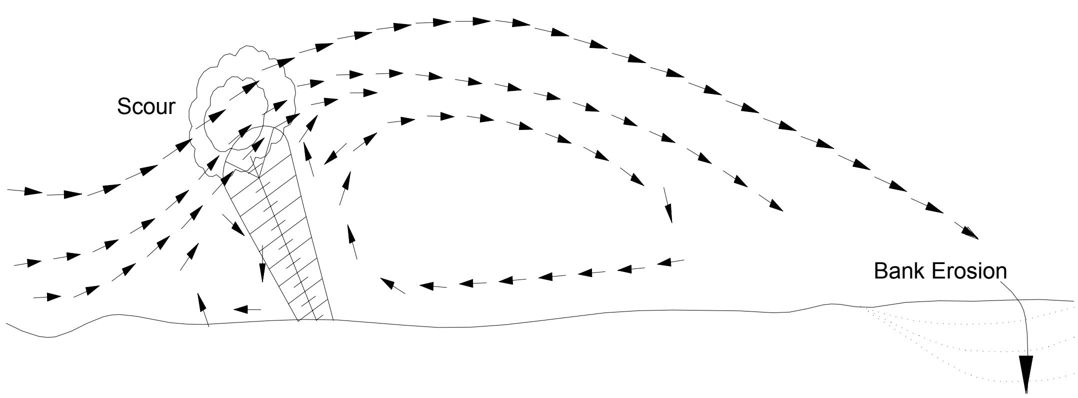
Types of groynes

(a) Classification Based on Orientation of Groynes

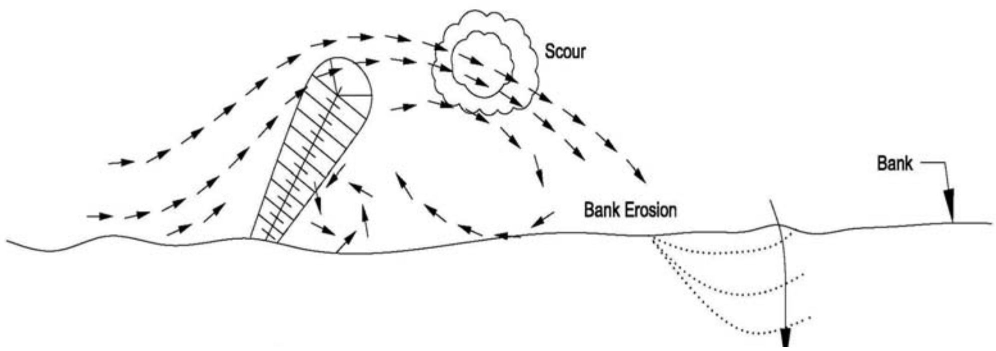
A groyne placed at right angle to the bank (perpendicular flow attack) is termed as *deflecting groyne*. Groynes inclined in upstream direction are called *repelling groynes.* It diverts the flow away from the structure. A*ttracting groynes* point downstream and attract the flow towards the structure's head and thus to the river bank (Fig. 3.14).

Groynes placed normal to the flow area may protect relatively small area. Groynes facing upstream deflect the flow away from the bank, and they are able to protect the bank areas upstream and downstream of themselves. Groynes facing downstream are not suitable for bank protection, since the current may attack the next groyne.

Groynes designed for bank protection should result in a maximum of deposition between the groynes. Single groynes provide only local protection. For that reason, several groynes are needed to form a groyne field and enlarge the stretch of protected river bank.



(a)



(b)

Fig 3.14 Single repelling and attracting groyne

(b) Classification Based on Shapes of the Groyne Heads

Due to blockage of the river flow, high local velocities and scour initiate and need massive scour protection measures. Fig. 3.15 shows different types of scour, notably *local scour*; *protrusion scour* and *constriction scour* at a straight groyne head which are not desirable. Constriction scour is observed when a straight groyne reduces the channel width by more than 10%. The constriction scour generally develops in the front of the head. At the downstream side of the head a local scour hole develops and if the flow at the upstream side of the groyne accelerates gradually a protrusion scour hole can develop at the upstream side of the head of the groyne. To improve the performance a large number of differently shaped groyne-heads have been tested over the last decades various type of groyne are shown in Fig.3.16.

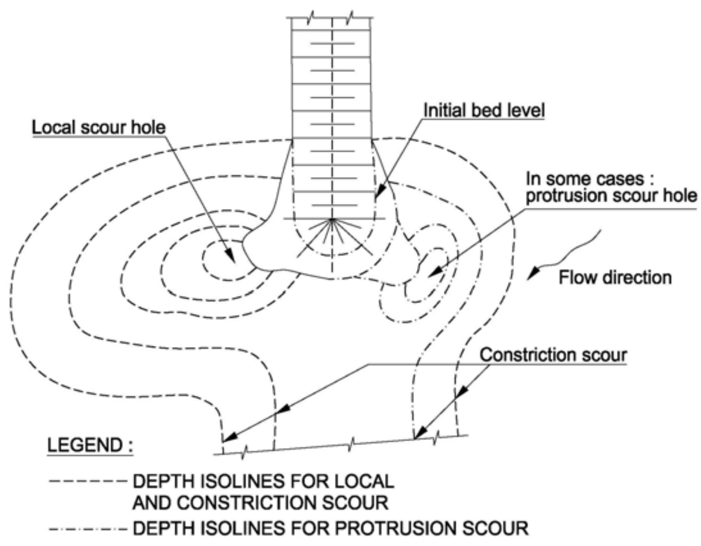


Fig 3.15 Scour around a deflecting groyne

Bell Head

Bell head (Fig 3.16b) provides extra volume of scour protection material and a gentler transition between groyne head and river bed. A bell head is often used if the direction of the approach flow is more or less fixed.

T-Head and L-Head

T-head or L-head groynes (Fig 3.16c, Fig 3.16d) are built when the direction of the approach flow upstream of the groynes can change due to morphologic development of the river channels.The aim is to guide the flow and reduce the scouring at the groyne head and to increase sediment depositions downstream from the groyne.

A T-head groyne has two wings with an equal length. A long upstream wing (L-groyne) can reduce the attack on the shank of the groyne. Examples of T-head groynes in Bangladesh are those constructed for Rajshahi town protection works in the Ganges with brick mattress cover layers and Sailabari groyne contructed in the Jamuna.

Hockey Stick Head

If a very strong attack on the head is expected, groynes with curved trunks known as hockey shaped groynes (Fig. 3.10e) can be adapted, which is a combination of a short L-head and a bell head. A reduction of the scour material may be achieved in this way as compared to T- and L-head groynes.

No general rules are available for determining the most convenient shape of the groyne head as well as the length of shank and it is advisable to undertake physical model tests for important protection works.

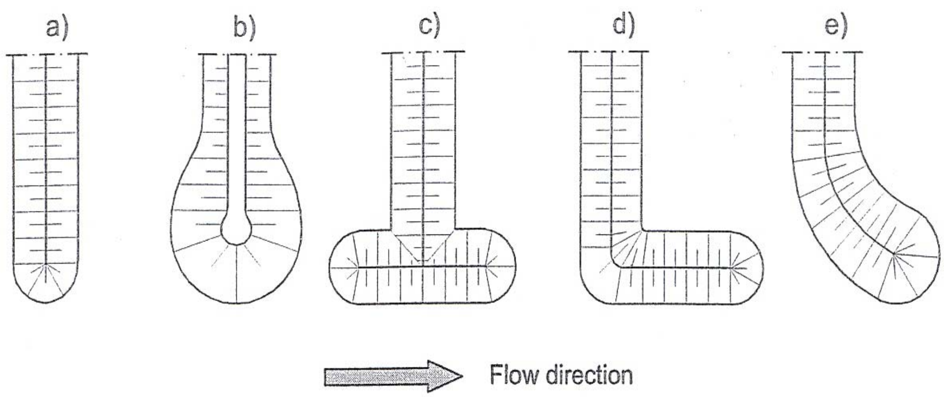


Fig. 3.16 Different types of groyne heads: (a) straight, (b) bell-head, (c) T-head, (d) L-head, (e) Hockey-head

(c) Classification Based on Permeability of Groynes

According to permeability the groynes can be classified as permeable and impermeable, the details of which is provided in Art. 9.5.

Composite Groyne

These groynes are combination of permeable and impermeable groynes (Fig 3.17). Generally, the country side part (on floodplain) of the groyne is made impermeable to reduce the flow velocity near the bank substantially whereas the riverside part is made permeable to reduce flow convergence and hence lower scour depth around the groyne head. Before implementing such groynes, proper study is required to check the intensity of reverse current near the bank due to impermeable part. Slope protection of the transition between permeable and impermeable parts, and bed protection around the permeable part are essential in such structures. Such groynes were constructed at Kamarjani in the Jamuna River.

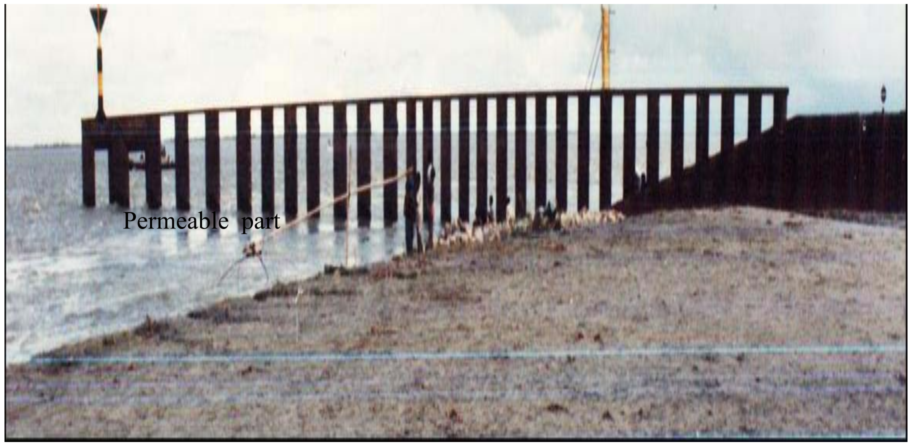


Fig. 3.17 Composite groyne constructed in the Jamuna

(d) Classification Based on Submergence of Groynes

These are two types: (i) unsubmerged groynes and (ii) submerged groynes.

Unsubmerged groynes

A groyne with crest level of the shank above high water level is called *unsubmerged groyne*. This type of groyne is normally constructed in Bangladesh. When impermeable and unsubmerged, the maximum intercepted hydraulic loads and scouring occurs around the head of the groyne, and maximum protection is required. Since the shank of the structure is not overspilled, no protection is required at the crest of the shank.

Submerged groynes

Submerged groynes are those where flow is allowed to pass over a considerable part of the shank at high water stage. The groyne crest is normally set below the bankfull level or with a sloping crest towards the river. Protection of the crest of groyne shank against erosion due to flow velocity and wave is required in this case. The concentration of flow around the groyne head is low to minimize the extent of maximum scour.

3.5.4.2 Revetments

Revetment is artificial roughening of the bank slope with erosion-resistant materials. A revetment mainly consists of a *cover layer*, and a filter layer. *Toe protection* is provided as an integral part at the foot of the bank to prevent undercutting caused by scour. The protection can be divided as *falling apron* or *launching apron,* which can be constructed with different materials, e.g., CC blocks, rip-rap, and geobags. Fig. 3.18 shows revetments and their different components.

Launching apron consists of interconnected elements that are placed horizontally on the floodplain and normally anchored at the toe of the embankment. The interconnected elements are not allowed to rearrange their positions freely during scouring but launch down the slope as a flexible unit. The *falling apron*, on the other hand, consists of loose elements (e.g., CC blocks, geobags, stones) placed at outer end of the structure. When scour hole approaches the apron, the elements can adjust their position freely and fall down the scouring slope to protect it.

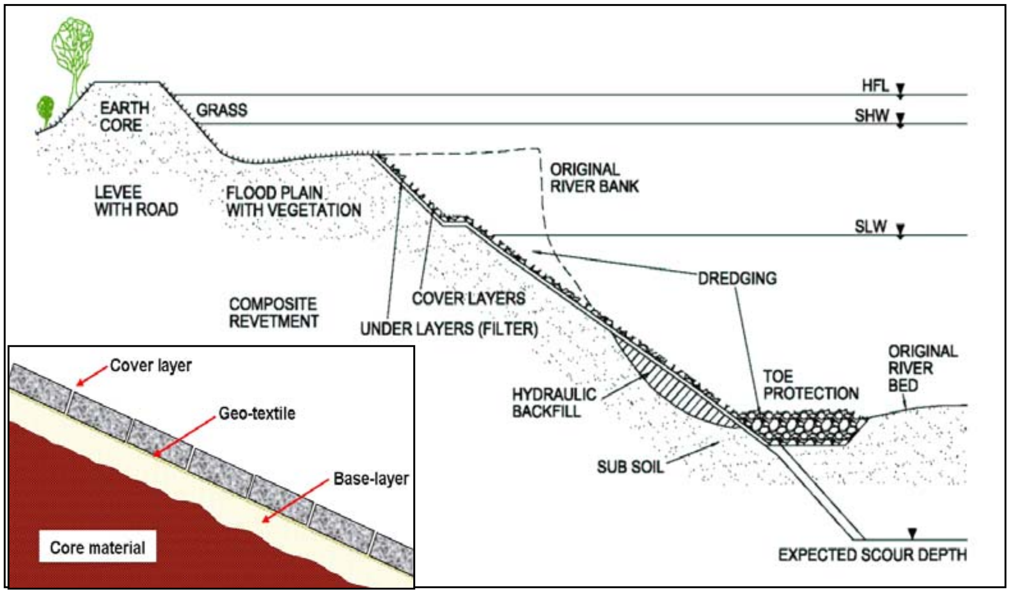


Fig. 3.18 Components of a revetment on riverbank

Revetment must have the following qualities and characteristics.

i) sufficient *weight* to keep the subsoil stable against uplift forces;
ii) sufficient *permeability* to prevent uplift forces;
iii) sufficient *thickness* with appropriate permeability at transition to reduce the rate of change of excess pore water pressure;
iv) *filter layer* to prevent migration of soil;
v) *flexibility* to accommodate movement within *subsoil* without loss of strength, and
vi) *stability* to withstand slipping on any construction plane.

Types of Revetment

Based on construction materials, revetment may be of different types: rock riprap, rubble riprap, wire enclosed rocks, pre-formed blocks, grouted rocks or paved lining etc and can be grouped as:

i) *Self-adjusting structures*- Includes all kinds of rip-rap protection using stones, concrete blocks, hand-laid bricks and concrete block layers without interlocking, slurry-filled bags, sand-filled geotextile bags etc (Fig. 3.19a).
ii) *Flexible structures*- Includes articulated blocks and slabs, wire mesh netting and other forms of mattresses, tubes, gabions and interlocking block layers (Fig. 3.19b).
iii) *Rigid structures*- Includes asphalt and concrete paving, concrete filled pillows, grouted mattress, grouted rip-raps, soil stabilization etc.

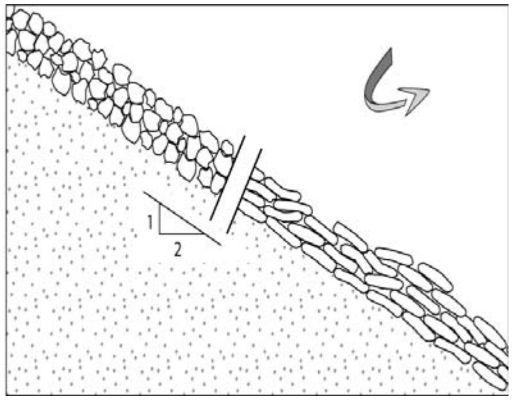


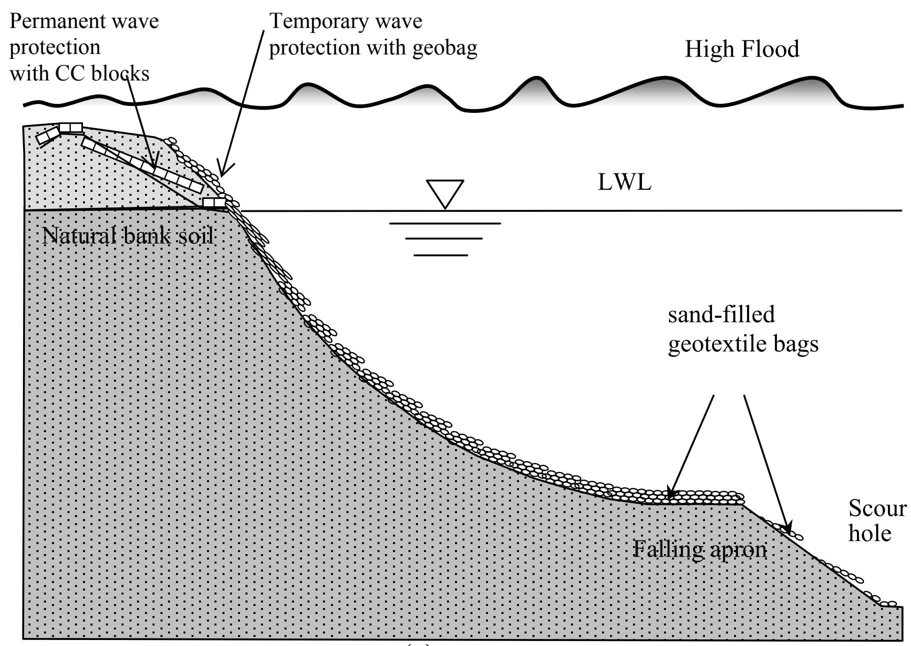
Fig. 3.19a Self-adjusting revetment: multi-layer riprap and geobags (left), loose cement concrete blocks (right)

(i)

(ii)

Fig. 3.19b Flexible revetments with interlocking cement concrete blocks (tongue-grove type) and cement grout mattress

Recently sand-filled geotextile bags (geobags), have been used in Bangladesh (Fig. 3.20). In this approach, the slope below low water level is protected with geobags. The slope above LWL is protected with cement concrete blocks mainly to resist wave action. The geotextile bags normally used are filled with sand (FM > 1.0) and weigh 78 kg or 126 kg. The advantages of this method are low cost and possibility of rapid deployment especially for structures with a limited design life. No additional filter layer is required as geobags have filter characteristics and retain the fine underlying subsoil.



(a)

(b)

Fig. 3.20 Geobag revetment (a) cross-section and (b) temporary protection

3.5.5 Criteria for Selection of Protection Structure

The selection of a specific type of a structure e.g., CC block revetment, geobag revetment, T-head spur or hard point, earthen-core groyne, RCC groyne etc. depends on a number of factors, such as

i) Loads and Protection
- Protection against flow velocity
- Protection against wave
- Resistance or sensitivity to partial failure

ii) Cost
- Initial implementation cost
- Damage repair and maintenance cost

iii) Socioeconomic implications
- Job opportunity and peoples participation
- Land use in area of structure
- Land reclamation procedure
- Pilferage of protection materials

iv) Ecological influence
- General
- Vegetation
- Fish

v) Technical influence of structure
- Navigation hazard
- Ghat operation (ship-shore transfer)

vi) Construction process
- Sensitivity of construction period
- Construction requirements
- Construction materials
- Construction in flowing water
- Preference to local contractors
- Labor intensive

vii) Adaptability, repair and maintenance
- Maintenance requirements
- Sensitivity
- Resistance or sensitivity to partial failure, ship collision

viii) Fund availability
- This is the most crucial point to be considered for assuring that planned measures can be timely executed. This is also crucial to frame construction phases and to avoid risk of modifications of the already completed design.

ix) *Land acquisition*
- Population displaced and their rehabilitation
- Land requirement
- Peoples behavior and sensitivity.
-

A multi-criteria analysis can be carried out for selection of a specific structure assigning a weightage or point to each of the above aspects. Weight of each group however, vary for individual implementation and require proper judgment.

3.6 Soil, Water, Structure Interaction

As explained earlier, the river banks, whether protected or unprotected, may be eroded in two ways: abrasion, i.e. removal of material from the surface of the bank; and geotechnical failure, i.e. the collapse of mass of soil into the channel. Abrasion is caused mainly by hydraulic loads. Geotechnical failures are mainly due to a reduction in the internal soil strength or an increase in the active forces (external loads, seepage gradients, etc).

The soil, water, structure interaction varies with the type of protection structure. For *loose rock* the load consists mainly of the velocity field, while the strength comes from the individual stone weight. For *placed bloc*k revetments, the interaction between top layer and filter layer and between the individual blocks play a role in the stability of the structure. For an *impervious asphalt or concrete layer*, the pressure fluctuations due to waves are too fast for lifting of the protection. The most important load is often due to more slowly varying water levels like tides and surges. When the water level outside rises slowly, the pressure inside also becomes high. When the level outside suddenly drops, e.g., after a storm, the pressure inside cannot follow easily the pressure outside and the layer can be lifted off.

(a) Hydraulic gradients within subsoil

External hydraulic loadings due to water level fluctuations and waves induce internal hydraulic such as fluctuations in water table, seepage flow and corresponding hydraulic gradients. For example, during a rapid drop of stage in river, the water table within the embankment soil cannot follow the change of the outer level immediately. As a result there is a lag in pore water pressure distribution within the bank subsoil compared with the pressure distribution on the surface.

(b) Uplift forces

If bank revetment is of relatively low permeability compared with the subsoil, it may be subjected to uplift forces caused by the outflowing water. When the uplift forces are not fully counteracted by the weight of revetment and friction between revetment and the underlying layer, then a sliding of revetments may appear.

(c) Internal stability of subsoil

The subsoil is internally stable if no internal migration of grains occurs. The most susceptible to internal erosion are non-uniformly graded soils with the coefficient of uniformity $C_u = D_{60}/D_{10} > 10 - 20$. For C_u between 10 and 20, the grain migration depends on hydraulic conditions. The strength of subsoil is characterized by a critical hydraulic gradient (I_c). A critical seepage velocity is also used sometimes. If the hydraulic gradient (I) within subsoil mass exceed I_c , grain migration begins. Usually, two basic types of gradients are considered: parallel and perpendicular to the interface (subsoil-filter). The latter is the most important to the subsoils protected by permeable revetments.

3.6.1 Common causes of Failures of Bank Protection

Causes which may lead to damage or failure may be classified as follows:

a. *Environment and Nature*
   i) river current and wind generated waves, causing direct erosion (scour) with outwash and undermining as well as abrasion and displacement of armour units;
      Impact forces caused by breaking waves on impermeable or poorly permeable revetments may cause liquefaction of granular subsoil which

may lead to failure of the revetments. Repeated wave loadings are conducive to internal instability of subsoil and fatigue of protective layers.

ii) outflanking of river channels due to morphological/hydrodynamic changes etc.; (Fig. 3.21)

iii) geotechnical instability and failures of the subsoil which are also influenced by natural calamities like earthquake, heavy rainfall, etc., resulting in liquefaction, flow slides etc.;

iv) flooding during high water stages causing soil saturation and overtopping, sloughing or sliding may occur due to seepage during receding water levels; (Fig. 3.22)

v) ground water movements causing piping;

b. Construction

i) faulty supply of protection materials or elements e.g. less in quantity or inferior in quality;

ii) poor workmanship due to:

- inadequate under water coverage, creating heaps in one area and gaps in other area;
- unavailability of proper equipments, or skilled labour
- improper and difficult execution of protection work due to turbid water, great depth and high flow velocity.

c. Human Factors

i) Inadequate investigations of natural conditions (hydrodynamic, morphological, topographic and subsoil surveys etc.);

ii) faulty design, e.g. by

- wrong interpretation or insufficient consideration of subsoil, hydrological and morphological conditions, loads, etc.;
- insufficient terminations (wing protection), resulting in outflanking;

iii) insufficient or no maintenance or repair of damages in time.

iv) excessive surface loading (surcharge);

v) damaging through

- vandalism;
- excavation too close to the structure, or

3.6.2 Revetment Failure

A revetment may fail due to:

i) instability of the cover-layer caused by external loads (current, waves etc.) or internal loads (e.g., due to pore water pressure),

ii) insufficient toe protection and instability or improper launching or falling behavior of the apron materials,

iii) sliding of cover layer over intermediate layer, and

iv) different micro- and macro-instability (described in Chapter 4) arising from geotechnical characteristics of soil and changed boundary conditions e.g., due to rapid scour development or water level changes.

Macro-instability is the deformations or displacements of relatively large soil masses that may occur gradually to very abruptly. *Micro-instability*, on the other hand, is related with the individual grains or protection elements on the slope.

The finer particle size and low relative density of bed and bank materials of the major rivers of Bangladesh are very much susceptible to micro- and macro-instability.

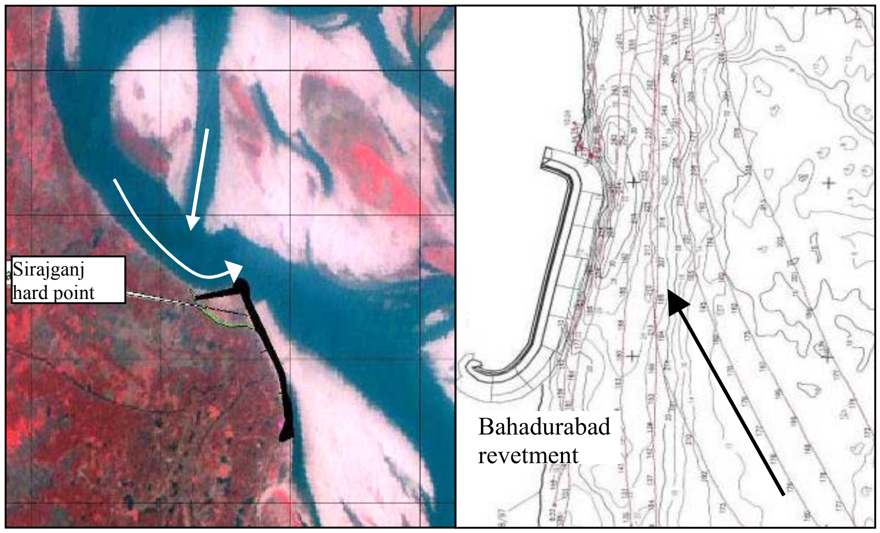


Fig. 3.21 Outflanking flow and angular flow attacks on revetments

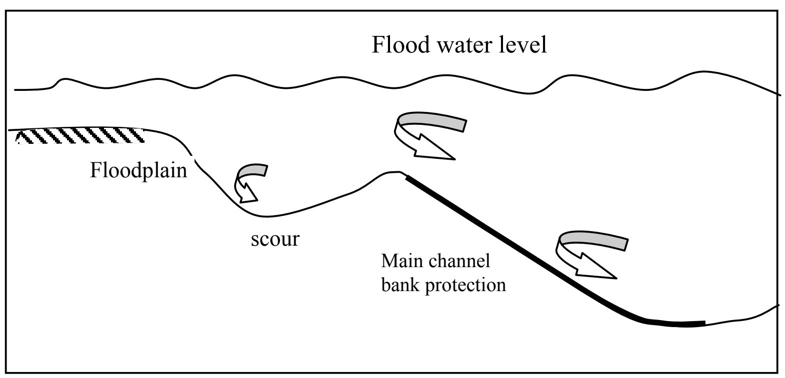


Fig 3.22 Erosion at floodplain level at flood flows may damage the structure

Different types of mechanism associated with the failure of a bank protected with revetment are shown in Fig. 3.23.

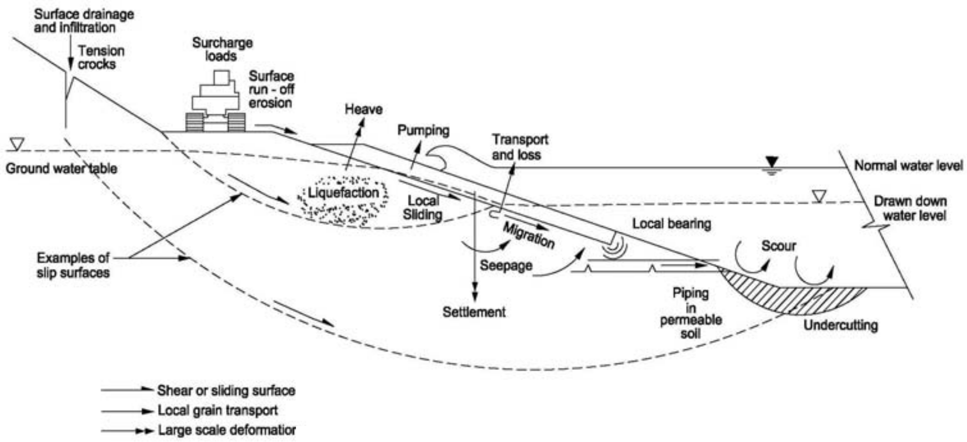


Fig. 3.23 Different Types of Subsoil Failure Mechanisms (PIANC, 1987)

Previous experiences with major revetment damages in the Jamuna River have identified several mechanisms of failures. Development of flow slides in the sub-aqueous slopes is one of the main mechanisms which occur normally during rapid development of the underwater slope by scouring. The loose micacious sandy soil of the Jamuna River is very much susceptible to liquefaction and flow slides due to its low relative density (in many places less than 50%). Once one or multiple flow slides occur under water, the slope retreated towards the structure. The falling apron may become ineffective to shield the developed scour profile at this stage and flow slides regresses underneath the apron.

Again, low integrity and lack of continuity between the cover layer and filter layer and also between the static part (main slope protection) and moving part (falling apron) of the structure during working condition may appear as a reason for major damages of the structures. Improper construction method and lack of monitoring and timely repair are human factors of failure in those cases.

3.6.3 Groyne Failure

The groyne fields constructed for bank protection are sensitive to morphological activities like shifting of river channels. Fig. 3.24 shows two examples. First one is the outflanking of the groyne field due to extreme shifting of the upstream channel in a single-thread meandering channel. The next one is the abrupt change in angle of flow attack in the groyne field due to shifting of a char and intensification of an oblique deeper channel flowing towards the bankline. In both cases severe damages of the groynes would occur and whole groyne field may become ineffective in few flood seasons if proper rehabilitation works are not undertaken rapidly.

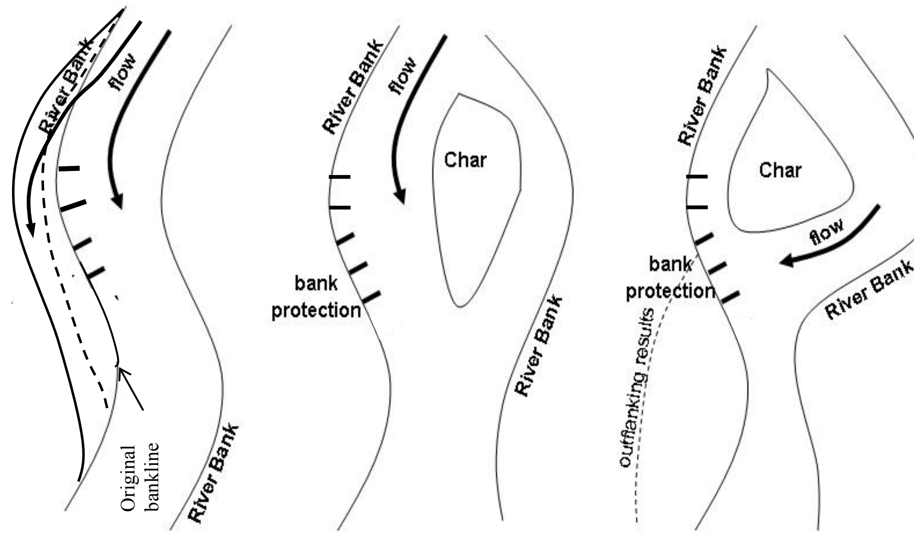


Fig. 3.24 Damages in groyne field: outflanking in a meandering channel and change in angle of attack due to shifting char in a braided river.

Impermeable earthen-core groynes

The failure mechanisms of conventional earthen-core impermeable groynes are almost similar to revetments when the groyne head and side slopes are normally protected employing same methods as revetment. With a nominal toe protection along the shank and traditional toe protection along head, which is the general approach, the experience may be quite different. Stages of damages and failure may be as follows:

i) At first stage, a scour hole is created due to generation of vortices at the groyne head.

ii) When the scour hole develops at very high rate and attains sufficient depth, a sequence of flow slides may occur. These flow slides may penetrate the bottom of the apron, which is unable to react rapidly to flow slides. Inadequate launching and coverage of the apron and sinking of apron materials may deteriorate the situation.

iii) At third stage local break-out and sliding of the lower part of the groyne head slope would occur and the filter materials may also disintegrate.

iv) At final stage, hydraulic action becomes dominant again when the cover layer is progressively dismantled and the exposed slope is eroded very rapidly.

v) Due to morphological changes of the river a parallel flow along the shank may develop and that may induce a scour hole at any location along upstream of the shank. The shank with nominal toe protection may fail detaching the head from the shank and finally invite collapse of the whole system.

Local slip failures are also observed in the groyne head and shank due to characteristics of sandy soil especially in the Jamuna. These require continuous monitoring and repair-maintenance of the structures.

Impermeable groynes are a type of dam constructed of stone, gravel, rock, sand, sandbags etc. The weight of the groyne causes compression of the subsoil resulting in settlements of the groyne. Possible failure mechanisms such as foundation failure and slope failure depend mainly on the type and condition of the subsoil. Penetrations, settlements and subsidence are to be expected.

RCC Spurs

The RCC spurs are built with a long earthen shank and a RCC head (~150 m long). The failure modes of these spurs are different compared to the earthen-core structures. The structure may fail due to failure of different components as follows (examples are in Fig. 3.25):

i) The transition between the earthen part and RCC part may fail by high hydraulic loads due to the oblique flows, diverted flows or back eddies generated by the head.
ii) The upstream slope of the earthen shank may fail due to parallel or oblique flow. The scour near the earthen shank may exceed beyond the design limit and slides may develop along the side slopes. The active channel then easily outflanks the RCC part.
iii) The RCC part may fail due to large scour development in front of the middle part of the vertical face. When the sheet piles are not sufficiently deep, the bottom of the structure may open forming large eddies and more scour downstream. The protection in the transition may also fail at this time.
iv) The apron placed around the RCC head mainly on unconsolidated soil (if RCC part is constructed on an island (char) after crossing a chute channel) may not follow the rules of falling apron as the stable slope for unconsolidated sandy soils is in the order of 1V:3.5H, whereas the protection element in the toe protection follows a slope of around 1V:2H or steeper.

Fig. 3.25 Partial (left) and full (right) damage of the earthen shank of RCC spurs

Permeable and Composite Groynes

The interaction between the piles of a permeable groyne and the subsoil is the same as with other pile structures exposed to horizontal loads from wave attack and currents. However, the possibility of cyclic load on the piles and the soil depending on the wave period must be taken into account.

The pile rows may fail due to rotation of an individual pile caused by insufficient pile embedment length. The piles may face such situations when excessive local scour occurs due to extra loading like increase of head difference between two sides of the pile rows due to unexpected floating debris (e.g., uprooted banana trunks, bamboo bundles, heap of partially decomposed water hyacinth etc.) accumulation and thus creating high-velocity underflows.

In case of composite groynes, the protection in the transition between the impermeable and permeable part may fail by sliding instigated by the deep scour hole developed at downstream side (Fig. 3.26). The deeper scour hole may be the result of direct high-velocity flow attack on the transition zone from a shifting oblique channel. Failure of piles near the transition may also result flow concentration and high scour.

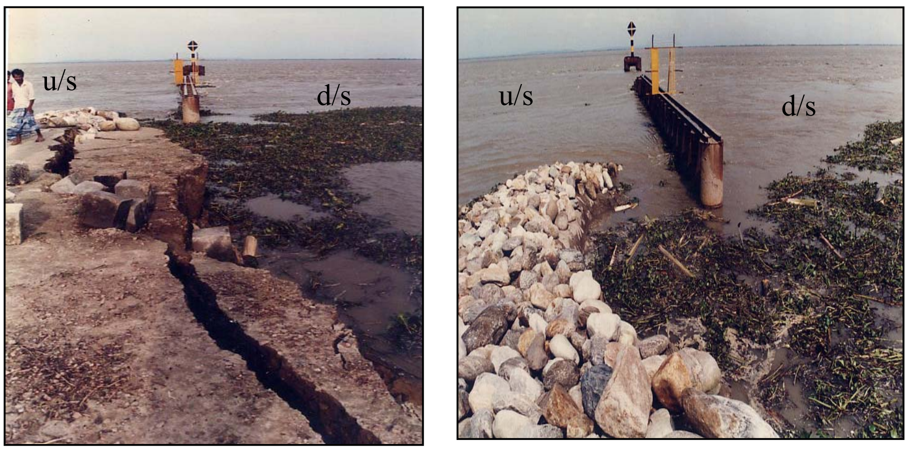


Fig. 3.26 Damages of the composite groynes

3.6.4 Riprap Failure Modes

Riprap failures are better classified by failure mode. Blodgett (1986) has identified classic riprap failure modes as follows:

- Particle erosion
- Translational slide
- Modified slump
- Slump

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results when the tractive force exerted by the flowing water exceeds the bank materials ability to resist movement.

Translational slides are usually initiated when the channel bed scours and undermines the toe of the riprap blanket. This could be caused by particle erosion of the toe material, or some other mechanism which causes displacement of the toe material.

Modified slump is the type of riprap failure referring to mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures is related to shear failure of the underlying base material that supports the riprap revetment. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material.

The size of the stones is a critical factor in the design. If the riprap is incorrectly sized, the stones will be plucked from the bed and moved downstream before the design velocity is reached.

Stone size also influences winnowing; erosion of the finer bed material through the voids between the riprap stones under the action of turbulence and seepage flows. Winnowing is more likely to occur in sand bed rivers than in coarser materials. A filter is often recommended to resist winnowing failure as the larger the stones the greater the gaps between adjacent stones and the easier it is for bed material to be winnowed from beneath. If the riprap layer is two to three stone diameter thick, the effect of this form of failure is minimal.

3.7 Remarks

River bank protection is a static or semi-static measure against a dynamic river system. The primary cause for bank erosion and failures of structures are rapid morphological development like angular flow attack outflanking resulting extreme hydrodynamic loading.

An organized monitoring system and an adaptive approach are required to manage bank erosion problem and maintenance of the implemented structures.

CHAPTER FOUR

Geological, Seismic and Geotechnical Aspects

4.1 Geological Aspects

Geologically Bangladesh can be divided into three broad physiographic regions. These are *Tertiary Hills*, *Pleistocene Uplands* and *Recent Plains*, which are also the major relief features of the country. The Recent Plains can be further subdivided into *Piedmont Alluvium*, *Floodplains* and *Tidal and Estuarine Floodplains* (Fig.4.1).

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 active Himalaya 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 oxidized, compacted and contain less water than the recent sediments. The latter are typically dark, loosely compacted and have high water contents and in some places with appreciable quantities of organic components.

The floodplain soils of the three main rivers Brahmaputra/Jamuna, Ganges and Padma consist of non calcareous alluvium soils, calcareous alluvium soils, gray and dark gray acid soils, gray and dark gray non saline soils, black, dark, gray and brown non acid soils (Fig 4.1). The oscillation zone of this river system is recently deposited with sediments which only contain sand, silt and clay soils. The latter are expected to be cohesive to some extent and exert a high 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 for the Ganges than the Brahmaputra/Jamuna.

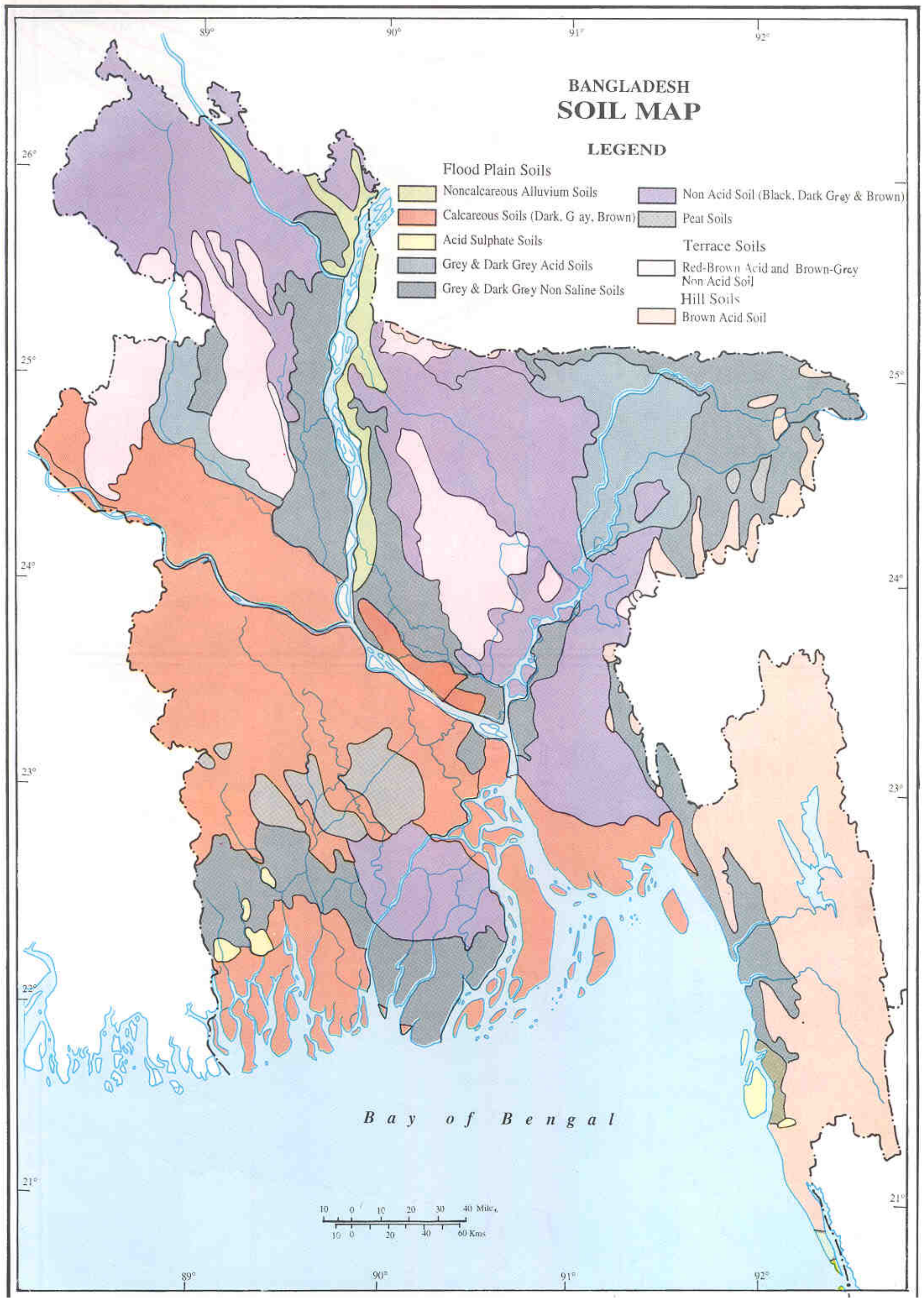


Fig 4.1 Bangladesh soil map

4.2 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. Recognizing this, the Committee of Experts on Earthquake Hazards Minimization has recommended that Bangladesh can be sub-divided into three seismic zones; Zone-I, Zone-II and Zone-III (Fig. 4.3). The report suggests the basic horizontal seismic coefficients of 0.075, 0.15 and 0.25 for Zone-I, Zone-II and Zone-III respectively.

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, 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 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.

Alluvial formations composed of loose sand and silt in saturated state need to be checked for ground liquefaction before any structure or structural measures are proposed.

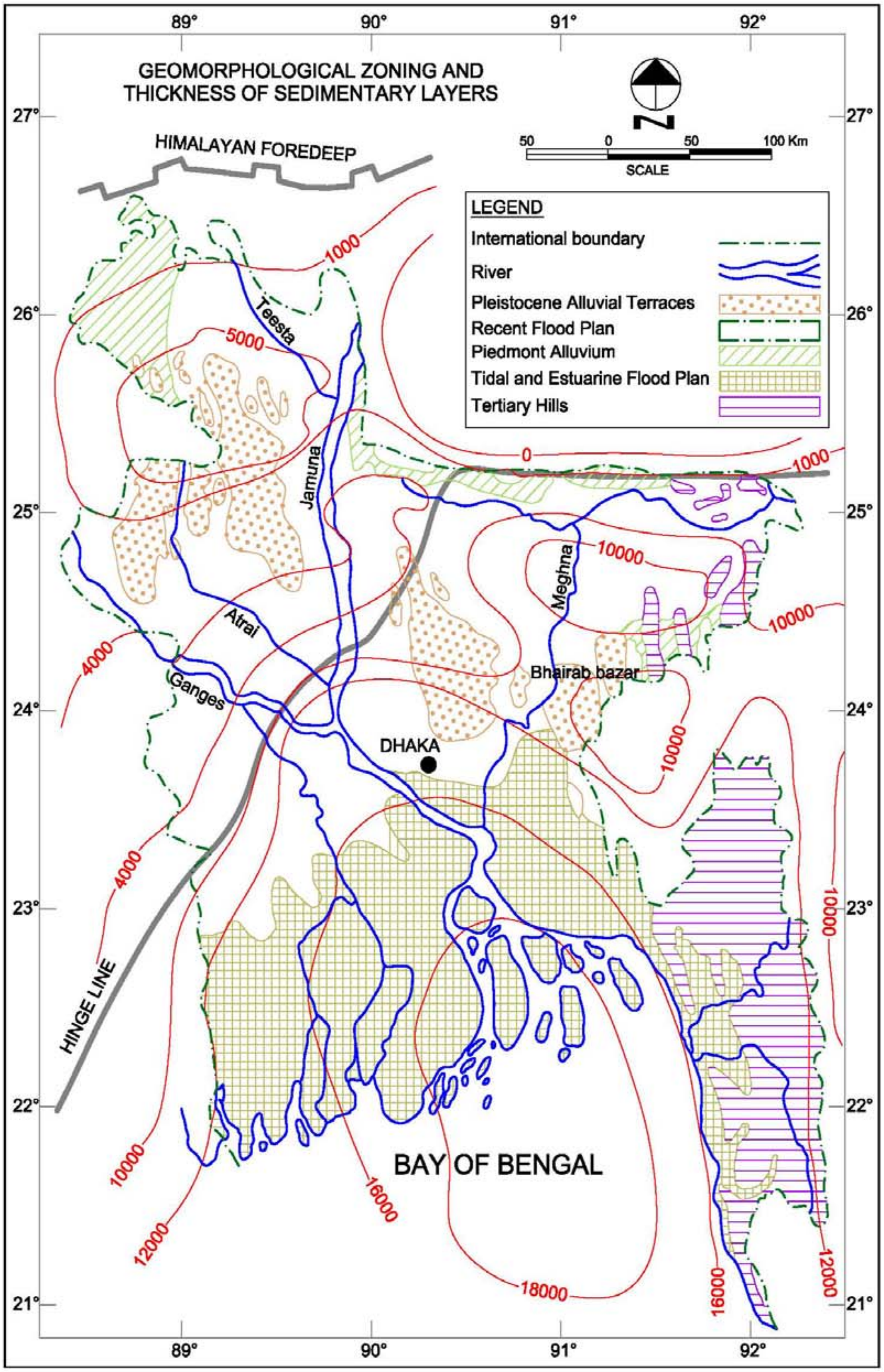


Fig.4.2 Geomorphologic Zoning and Thickness of Sediment Layer.

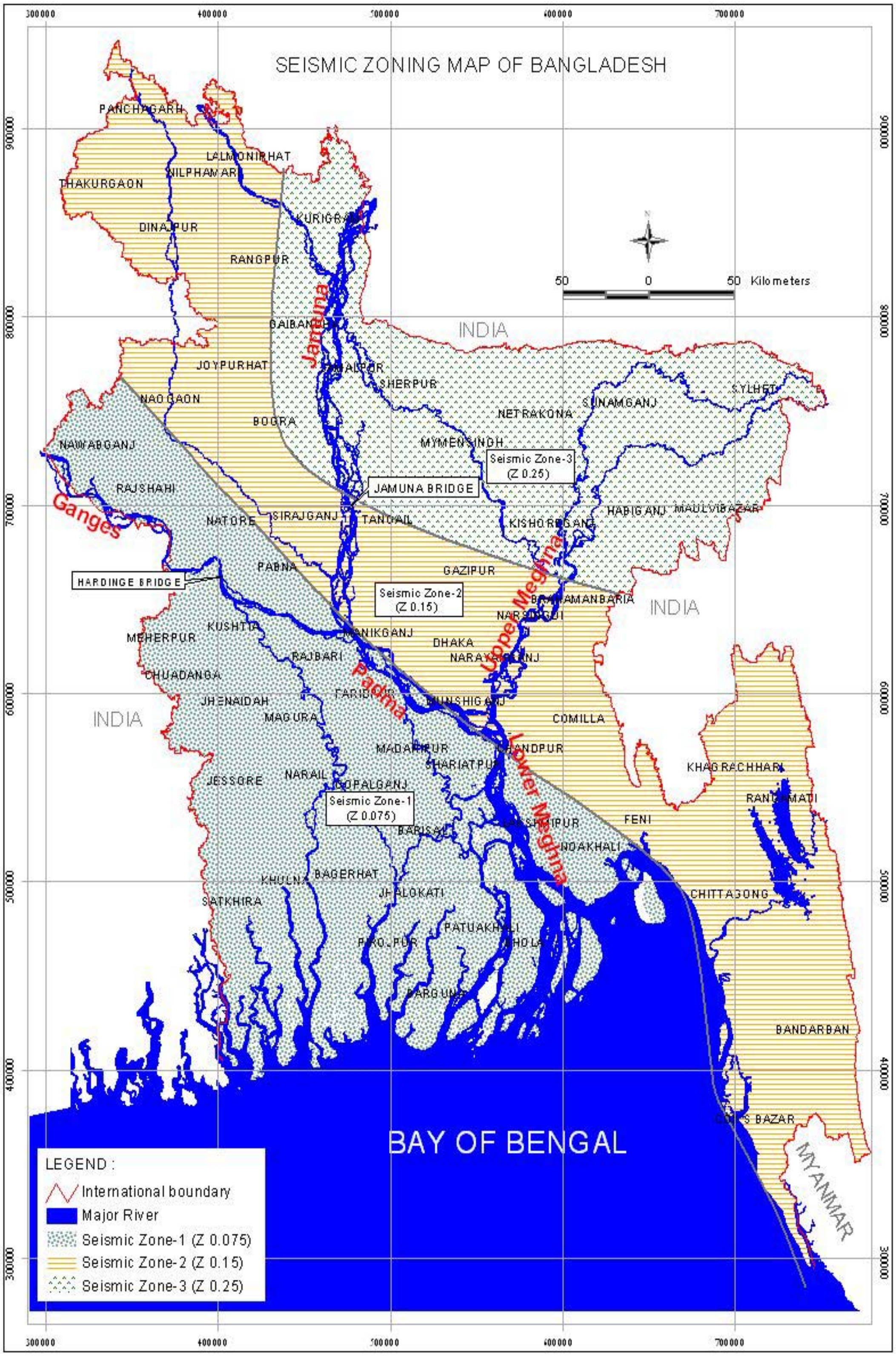


Fig.4.3 Seismic Zoning Map of Bangladesh (Source : BNBC 1993)

4.3 Geotechnical Aspects

Proper attention to the geotechnical aspects is needed for a bank protection or river training works to prevent failure. The manner in which failure can occur can be related to a failure mechanism. Major failure mechanisms that can develop on a river bank and associated structures are outlined below.

1. Slope failure : Macro and micro instability
   Controlling factors:
   - internal shear strength
   - excess pore-pressure
   - liquefaction
2. Ground settlement
   Controlling factors:
   - densification of the loose deposit
   - consolidation of soft subsoil layer
   - squeezing of very soft subsoil layer
   - loss of material from sub-layers due to piping
   - collapse of underground cavities
3. Slope and toe erosion and undercutting
   Controlling factors:
   - surface protection (block, geobag, geotextile)
   - filter gradation

4.4 Geotechnical boundary conditions and data collection

Depending on the functional requirements, river bank and river training structures should be designed and constructed to withstand a combination of actions induced by waves, currents, fluctuations in water levels, seepage pressure, cyclic loads (earth quake or wave induced) etc. These actions, including the self-weight of the structure, have to be transferred to the subsoil in such a way that:

1. The deformations of the structure are acceptable

and 2. The probability of loss of stability is sufficiently low.

The transfer of actions to the subsoil involves (time-dependent) changes in soil stresses, both within the structure and in the soil layers. Due to these stress increments, the soil layers and, as a consequence, the structure may deform vertically and horizontally or may even lose its stability. This deformation behaviour depends not only on the external loadings but also on the geometry (slope steepness), the structure's weight and the permeability, stiffness and shear strength of the soil layers. In fact, the performance of a waterfront structure depends to a large extent on the interaction between the structure and the subsoil. For this reason, a good knowledge of the main geotechnical properties of the soil layers is needed for design of bank protection and river training structures. The soil data (or parameters) relevant to geotechnical aspects of design are to be collected through soil investigation program.

4.5 Soil-investigation programme

A proper specification of a soil-investigation programme should provide the following :

- name of soil data or parameters that have to be collected

- level of reliability or precision with which the above data should be obtained
- locations (number, spatial distribution and depths)
- site investigation techniques, laboratory tests and equipments
- time frame to carry out the programme

and
- define responsibility for the contracting work in the Field, the Laboratory Tests and the interpretation of the test results

The following conditions play a major role in preparing the soil investigation programme:

- The global structural requirements of the project, including location, layout, and nature and extent of structure
- The availability of existing geotechnical data
- The knowledge and experience of geotechnical experts
- The detailed structural requirements expressed in terms of geotechnical limit states (allowable stability and deformations), including the type of loading schemes and failure mechanisms
- The applicability (including the terrain accessibility) of the available investigation tools and qualified personnel
- Last (but not least) the boundary conditions stipulated by the controlling authority in terms of time and money

A high-quality investigation must be economically efficient in the sense that the cost of the soil investigation must be justified by building up additional knowledge of the geotechnical behaviour of the structure. It is emphasised that in the case of over-water site investigations the costs of operating equipment (vessels or platforms) forms the major part of the total cost of geotechnical investigations.

For large projects, programming and execution of soil investigations is a growing process, in which the need for additional soil data depends on the previously collected data and on the phase of the design stage. Three steps of a soil-investigation programme can generally be distinguished and are related to the overall design process :

1. Desk study : collecting available topographical, geological, hydrological and geotechnical data for an initial appraisal of the boundary conditions

2. Preliminary soil investigations : obtaining global information on soil data over a wide area aiming at recognising the main geotechnical problems, making a proper choice of the location of the structure and allowing alternative designs to be considered and a preferred solution to be selected

3. Final soil investigation : obtaining detailed soil data defining specified geotechnical parameters in order to prepare the final design and construction method of the structure

4.5.1 Desk studies

The purpose of desk study is to gather and compile available data. For major projects a good geological knowledge is of the greatest importance for an understanding of the geophysical and hydro-geological conditions. The most important geological aspects are:

• Geological stratification, formation and history
• Groundwater regime
• Seismicity

The geological map may be an important source for the above information. In addition to geological maps or descriptions the following data is also helpful:

• Topographical and bathymetrical maps;
• Groundwater maps (on land site);
• Climatological maps;
• Data on water levels, waves and currents;
• Local maps from other organisations;
• Historical data on observed geotechnical failure mechanisms;
• Published geotechnical data relating to the area or geological formations.

4.5.2 Preliminary soil investigations

In many cases there may be no or insignificant available soil data. In such a case, where little can be determined of the subsoil conditions from the desk study a preliminary soil investigation must be performed.

The preliminary soil investigation should be designed to provide a proper insight into the subsoil conditions in order to identify the main geotechnical problems that may influence the location and the global design alternatives for the structure. The investigation should therefore permit the type and thickness of all soil strata to be established, both soft and compressible layers and harder layers. In addition, during the preliminary investigations, information will be gained on the possible sources of construction materials such as sands, gravel and rockfill.

Depending on the data available from the desk study and the global functional requirements of the bank structure, a preliminary soil-investigation programme will generally consist of stratigraphical surveys that may comprise of a small number of borings (with disturbed sampling} and penetration tests.

After preliminary design alternatives have been selected, detailed soil investigations have to be carried out to prepare the Final design.

4.5.3 Detailed soil investigations

The detailed soil investigation programme should provide sufficient information on those soil parameters which are decisive for the selection of the (economic) optimum design, construction and maintenance alternative.

The type of soil properties to be collected, their reliability and the locations must be directly related to the functional requirements of the proposed design alternatives. In Table 4.1 a summary is given of the main soil parameters to be determined for five types of geotechnical failure mechanisms, including shallow and deep-seated slip failure, liquefaction, dynamic failure, excessive settlements and filter erosion.

Table 4.1 Required soil data for the evaluation of geotechnical limit states (Reproduced from Table 29 of CIRIA Special Publication 83/CUR Report 154)

Geotechnical limit states					
Macro Instability			**Macro failure**	**Micro Instability**	
Slip failure	**Lique-faction**	**Dynamic failure**	**Settle-ments**	**Filter /erosion**	**Geotechnical information**
A	A	A	A	A	Soil profile
A	A	A	A	A	Classification/ grain size
A	A	A	B	A	Piezometric pressure
B	B	B	A	A	Permeability
A	B	B	A	B	Dry/ wet density
-	A	B	-	-	Relative density, porosity
A	B	B	-	C	Drained shear strength (c', ϕ')
A	-	-	-	C	Undrained shear strength (S_u)
B	-	-	A	-	Compressibility
A	-	-	A	-	Rate of consolidation (c_v)
B	B	A	A	-	Moduli of elasticity (G, E)
B	A	A	A	-	In situ stress (σ')
-	A	B	A	-	Stress history (OCR)
B	A	A	B	-	Stress/strain curve

A : Very important
B : Important
C : Less important

The reliability of the data to be collected is mainly a matter of economics. Accurate description of the geotechnical failure mechanisms (at higher costs of soil investigations) may lead to a optimised design at lower construction costs. Once the relevant soil parameters have been identified, the suitable soil tests (laboratory and field) can be selected.

Although there is a great variety of testing methods, each with its own merits and restrictions Table 4.2 may give a first guide to selecting appropriate site-investigation program. In-situ test methods may not be able to provide all the desired information, and therefore may need to be supplemented by laboratory tests.

In Bangladesh SPT is common rather than CPT. SPT provides both dynamic penetration resistances (number of blow counts) and disturbed soil samples. CPT has significant advantages where soft soil conditions prevail.

For soil exploration in water a temporary fixed platform or floating facility (such as boats or pontoons) may be used. The cost of exploration in water is much higher than that on ground and becomes more difficult with the increase in depth, stronger current and higher waves.

Table 4.2 Test methods and their perceived applicability (modified from Table 30 of CIRIA Special Publication 83/CUR Report 154)

	Field tests				Laboratory test	
Information/ parameter	**Cone penetrat-ion test (CPT)**	**Standard penetration test (SPT)**	**Vane shear test (VST)**	**Moni-toring well**	**Disturbed sample**	**Undis-turbed sample**
Soil profile	A	A	B	----	A	A
Classification	B	B	B	----	A	A
Piezometric pressure	----	----	----	A	----	----
Permeability	----	----	----	C	C	A
Density	C	C			C	A
Relative density	B	B	----	----	----	A
Friction angle	B	B	C	----	B	A
Undrained shear strength	B	C	A	----	----	A
Compressibility	C	C	----	----	----	A
Rate of consolidation	----	----	----	----	----	A
Stress history OCR	C	C	B	----	----	A
Stress-strain curve	----	----	----	----	----	A
Ground conditions						
Sand	A	A	----	----	A	C
Silt	A	B	B	----	A	A
Clay	A	C	A	----	A	A
Peat/organic soil	A	C	B	----	A	A

A - High applicability
B - Moderate applicability
C - Limited applicability

4.5.4 Laboratory tests

Laboratory tests required for assessment of design parameters may be broadly classified into two types

(1) Classification test

Grain size distribution – For clean sand sieve analysis will be sufficient. For soils containing significant amount of fine particles (silt and/or clay) only hydrometer analysis or both sieve and hydrometer analysis may be relevent.

Atterberg limits – Required for cohesive soil

Permeability tests – For cohesionless soils usually undistrubed samples can not be obtained. Therefore permeability is determined from samples recompacted at varying density. For cohesive soils, permeability tests are performed on undisturbed samples. Since horizontal and vertical permeability usually differs, the test should be performed on both of horizontal and vertical samples. Permeability test can also be determined indirectly from the results of consolidation tests.

(2) Stress-strain test

Consolidation test – If there is soft compressible layer (such as clay or silty/sandy clay) then this test is relevant to obtain the parameters Compression index (C_c), Re-compression index (C_r) and Coefficient of consolidation (C_v) which are needed for settlement computations. Undisturbed sample is required to carry out this test.

Unconfined compression test – This test is relevant to medium stiff to stiff cohesive soils and is performed on undisturbed samples to obtain unconfined compressive strength q_u (or cohesion c_u). This test may not be applicable to soft clay because of greater sample disturbance.

Direct shear test – Applicable to both cohesive and cohesionless soil and provides shear strength parameters c and ϕ in drained or undrained condition. However, this test has inherent limitations such as a fixed shear plane (which may not be the weakest plane).

Triaxial test – The triaxial test is a better shear test, though relatively difficult, compared to direct shear test and requires much expertise. It allows determination of the shear strength parameters of soil under different drainage and stress conditions (such as different confining pressure). There may be Consolidated Drained (CD) test, Consolidated Undrained (CU) test, and Unconsolidated Undrained (UU) test. The choice of the type of tests depends on the permeability of the soil, rate of construction etc.

4.6 Geotechnical Limit states for a slope

The transfer of external loads through the structure to the subsoil involves changes in soil stresses (pore water pressures and effective stresses) in the soil layer and in the structure itself. Especially in soft soil as in Bangladesh the stress changes may gradually occurr over a long period of time. Due to these changes in soil stresses the underlying and adjacent soil layers will deform vertically and horizontally while the shear strength of the soil may be reduced. As a consequence any structure built on top of the soil layers will deform too or may even loose its stability. This applies to both the design conditions and the loadings during the construction period of the structure (Pilarczyk, 2000).

The changes in soil stresses and the associated deformations not only depend on the hydraulic loadings, but on the geometry (steepness of slope), the weight of structure and on the permeability, stiffness and shear strength of the underlying structure and soil layers as well. For this reason the design of hydraulic structures has to be based on an integral approach of the interaction between the structure and the subsoil. A good knowledge of the main geotechnical properties of the soil layers and the construction materials is therefore required.

The main geotechnical limits that should be investigated in the design of the revetment structures are (Pilarczyk, 2000):

- macro-instability of slopes due to failure along circular or straight sliding surfaces
- micro-instability of slopes caused by the seeping out of groundwater

- liquefaction caused by erosion or by cyclic loading (earthquakes, wave action, dredging etc.)
- erosion of revetments at the under water slopes due to unstable filters
- erosion of revetments at the under water slopes due to local failure of top layer of protection elements

4.6.1 Macro stability of slope

The material composing a slope has a natural tendency to slide under the influence of gravitational and other forces (eg. seismic activity) which is resisted by the shearing resistance of the material. Instability occurs when the shearing resistance is not enough to counterbalance the forces tending to cause movement along any surface within a slope. A slope that has been stable for many years may suddenly fail due to one or several of the following main causes.

(a) external disturbances in the form of cutting / dredging / scour near toe.
(b) seismic activity
(c) increase in pore water pressure within a slope (may be due to rise in water level or exceptional rainfall)
(d) progressive decrease in shear strength of the slope material
(e) progressive change in the stress field within a slope (may be due to subsidence of foundation soil, piping or internal erosion, movement of soil particles)

It can be stated that in case of slope failure the moved masses are so voluminous that even multi-layer of protection will not be able to stop the failure. Therefore the primary objective of all protection measures is to prevent slope failure (Fedinger, 2006).

4.6.2 Phenomena Governing Macro Stability of Slope

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

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

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

- By reducing or eliminating cohesive strength
- By producing pore water pressure which reduce effective stresses, thereby lowering effective strength
- By producing horizontally inclined seepage forces which increase the overturning moments and possibility of failure
- By lubricating failure planes after occurring small initial movement

- By supplying an excess of fluid that becomes trapped in soil pores during earthquakes or other severe shocks, leading to liquefaction failures.

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

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

- The body of a developed natural slope may become overstep when ground water flow due to subsequent falling water levels in the river results in a reduction of effective stresses due to uplift forces. Simultaneous reduction of shear resistance will then reduce its safety against sliding and may ultimately, render a slope oversteep.

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

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

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

4.6.3 Slope stability analysis

Various techniques of slope stability analysis are found in literature. Among them three methods that are applicable to river banks in Bangladesh are presented here.

(1) Ordinary Method of Slices
(2) Simplified Bishop Method of Slices
(3) Translational Failure Analysis – (applicable to layered soil)

4.6.3.1 Ordinary Method of Slices

This method is also known as Swedish Circle Method or Fellenius Method. A slope can be analysed for both drained and undrained conditions. A circular failure surface (about an arbitrary center of rotation) is assumed and the potential sliding mass is divided into arbitrary number of imaginary vertical slices (refer to Fig. 4.4(a) for the ith slice and definition of terms). It is assumed that the forces acting upon the sides of any slice have zero resultant in the direction normal to the failure arc for that slice. Factor of safety for a potential failure surface is determined as :

$$FS = \frac{\sum\left[\left(c_i * l_i\right) + \tan\varphi_i * \left(W_i * \mathrm{Cos}\theta_i - u_i * l_i\right)\right]}{\sum W_i * \mathrm{Sin}\theta_i}$$

Analysis is carried out for several circular failure surfaces and center of rotation to find the least factor of safety.

For homogeneous soil cohesion (c) and angle of internal friction (ϕ) are constant throughout the soil mass and therefore the above equation reduces to

$$FS = \frac{c * \sum l_i + \tan\varphi * \sum (W_i * Cos\theta_i - u_i * l_i)}{\sum W_i * Sin\theta_i}$$

For a slope consisting of cohesive soil, undrained analysis is appropriate for which

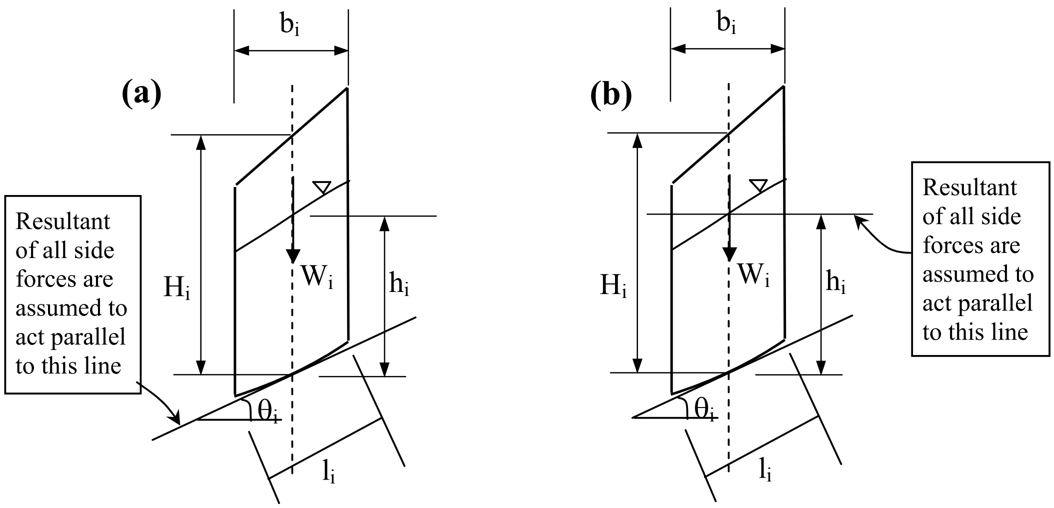
$$FS = \frac{\sum s_u * l_i}{\sum W_i * Sin\theta_i}$$

Fig.4.4a shows the determination of factor of safety by Ordinary Method of Slices for a single failure surface for a slope in homogeneous soil (Fig.4.4b). The steps of analysis are outlined below.

1. The cross-section of the slope is drawn to a scale.
2. An arbitrary circular failure surface is drawn by selecting a center of the circle.
3. The phreatic line is drawn on the cross-section (The phreatic line is the top surface of the flowing water through the slope due to difference in water head. On the phreatic line water is in atmospheric pressure).
4. The circular cross-section is divided into vertical slices and the slices are numbered (column 1).
5. The width (b_i) of each slice is estimated and recorded (column 2).
6. The average height (H_i) of each slice is estimate and recorded (column 3).
7. The angle θ_i for each slice is estimate and recorded (column 4).
8. The length l_i of the assumed failure surface cut by the each slice is estimated and recorded (column 5). (the length l_i may be estimated as $b_i/Cos\theta_i$)
9. The average height of water h_i in each slice above the slip surface is estimated and recorded (column 6)
10. Total weight of each slice (W_i) is calculated (column 7). $W_i=b_i*H_i*\gamma$ where γ = Unit wt of soil.
11. For each slice $W_i*Sin\theta_i$ is calculated (column 8)
12. The water pressure u_i is determined as $u_i = h_i * \gamma_w$ (column 9)
13. For each slice $W_i*Cos\theta_i - u_i*l_i$ is calculated
14. Σl_i, $\Sigma(W_i*Sin\theta_i)$ and $\Sigma(W_i*Cos\theta_i - u_i*l_i)$ for each slice are calculated
15. FS is calculated as

$$FS = \frac{c * \sum l_i + \tan\phi * \sum (W_i * Cos\theta_i - u_i * l_i)}{\sum W_i * Sin\theta_i}$$

Note : The term ($W*Cos\theta - u*l$) may become negative implying negative effective normal stress (i.e. tension) on the base of a slice which is un-realistic and unacceptable. This can happen for any slice with small W or large u or both. In practice the term may be taken as zero when its value for any term become negative.



Cohesion (c_i) and angle of internal friction (ϕ_i) act along the failure surface of the slice to resist sliding

Fig.4.4 Freebody of a slice for analysis by (a) Ordinary Method of Slices and (b) Simplified Bishop Method of slices.

4.6.3.2 Simplified Bishop Method of Slices

In this method it is assumed that the forces acting on the sides of any slice have zero resultant in the vertical direction (Fig.4.3(b)). This leads to the following expression for factor of safety of an arbitrary circular failure surface.

$$FS = \frac{\sum \left[\frac{c_i * b_i + (W_i - u_i * b_i) * \tan\varphi}{M_i(\theta)} \right]}{\sum W_i * \mathrm{Sin}\theta_i}$$

where $M_i(\theta) = Cos\theta_i * \left[1 + \frac{Tan\theta_i * Tan\phi_i}{FS} \right]$

Determination of factor of safety for an arbitrary failure circle require trial and error because FS appear on both sides of the equation.

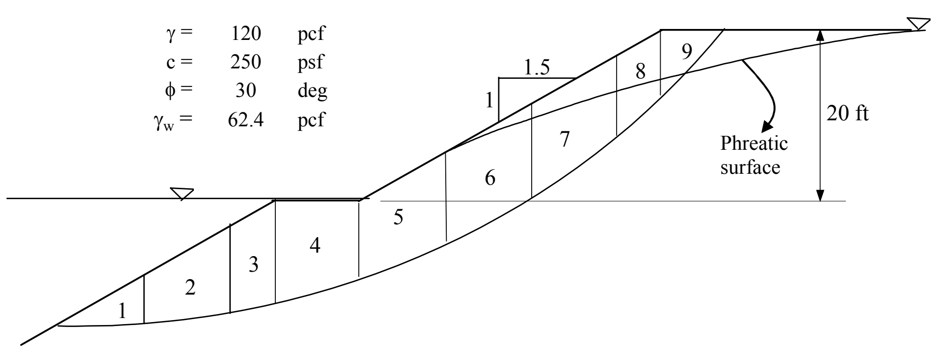
The slope shown in Fig.4.4a is analysed by Bishop Simplified Method of Slices in Fig.4.4b and the steps of analysis are outlined below.

1. The cross-section of the slope is drawn to a scale.
2. An arbitrary circular failure surface is drawn by selecting a center of the circle.
3. The phreatic line is drawn on the cross-section (The phreatic line is the top surface of the flowing water through the slope due to difference in water head. On the phreatic line water is in atmospheric pressure).
4. The circular cross-section is divided into vertical slices and the slices are numbered (column 1).
5. The width (b_i) of each slice is estimated and recorded (column 2).
6. The average height (H_i) of each slice is estimate and recorded (column 3).

7. The angle θ_i for each slice is estimate and recorded (column 4).
8. The average height of water h_i in each slice above the slip surface is estimated and recorded (column 5)
11. Total weight of each slice (W_i) is calculated (column 6). $W_i=b_i*H_i*\gamma$ where γ = Unit wt of soil.
11. For each slice $W_i*Sin\theta_i$ is calculated (column 7)
12. The water pressure u_i is determined as $u_i = h_i * \gamma_w$ (column 8)
13. For each slice $c_i*b_i + (W_i– u_i*b_i)*\tan\phi$ is calculated (column 9).
14. $M_i(\theta)$ is calculated for an assumed factor of safety (FS) (column 10).
15. $\{c_i*b_i + (W_i– u_i*b_i)*\tan\phi\} / M_i(\theta)$ is calculated for each slice (column 11).
16. $\Sigma(W_i*Sin\theta_i)$ and $\Sigma[\{c_i*b_i + (W_i– u_i*b_i)*\tan\phi\} / M_i(\theta)]$ for each slice are calculated
17. FS is calculated. If this FS differ from the assumed FS (step 14) then steps 14 and onward are repeated using another assumed FS. Iteration is continued until the calculated FS is close to assumed FS.

Note : If the term (W – u*b) for any slice become negative it should be taken as zero.

Ordinary Method of Slices

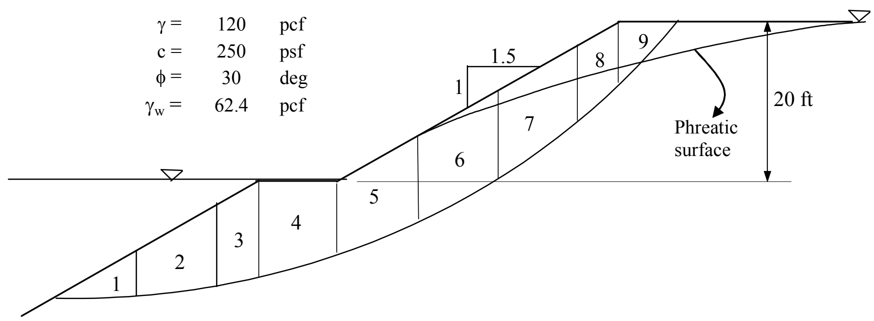


Slice no.	Width b_i (ft)	Avg. Height H_i (ft)	Angle θ_i (deg)	l_i (ft)	Water head h_i (ft)	Weight W_i (Kip)	$W_i*\sin\theta_i$ kip	u_i (kip/ft)	$W_i*Cos\theta_i-u_i*l_i$ (kip)
Col.1	Col.2	Col.3	Col.4	Col.5	Col.6	Col.7	Col.8	col.9	col.10
1	10	3	3.5	10.02	15.20	3.60	0.22	0.95	-5.91
2	10	8.2	7.5	10.09	14.30	9.84	1.28	0.89	0.76
3	5	11.5	12	5.11	12.80	6.90	1.43	0.80	2.67
4	10	10.6	15.5	10.38	10.80	12.72	3.40	0.67	5.26
5	10	10	22	10.79	9.80	12.00	4.50	0.61	4.53
6	10	11.3	27.6	11.28	10.90	13.56	6.28	0.68	4.34
7	10	10.7	35	12.21	7.60	12.84	7.36	0.47	4.73
8	5	8.5	44	6.95	4.00	5.10	3.54	0.25	1.93
9	7	4.5	49	10.67	0.00	3.78	2.85	0.00	2.48
Total =				87.49			30.88		26.70

$$FS = \frac{0.25*87.49 + \tan 30^0 * 26.7}{30.88} = 1.21$$

Fig.4.4a Example of analysis by Ordinary Method of Slices

Simplified Bishop Method of slices



γ = 120 pcf ϕ = 30 deg

c = 250 psf γ_w = 62.4 pcf Assumed factor of safety, FS = 1.15

Slice no.	Width b_i (ft)	Avg. Height H_i (ft)	Angle θ_i (deg)	Water head h_i (ft)	Weight W_i (Kip)	W_i*$\sin\theta_i$ kip	u_i (kip/ft)	c_i*b_i + (W_i-u_i*b_i)*tanϕ (kip)	$M_i(\theta)$	Col.9/Col.10
Col.1	**Col.2**	**Col.3**	**Col.4**	**Col.5**	**Col.6**	**Col.7**	**Col.8**	**Col.9**	**Col.10**	**Col.11**
1	10	3	3.5	15.20	3.60	0.22	0.95	-0.90	1.03	-0.87
2	10	8.2	7.5	14.30	9.84	1.28	0.89	3.03	1.06	2.87
3	5	11.5	12	12.80	6.90	1.43	0.80	2.93	1.08	2.70
4	10	10.6	15.5	10.80	12.72	3.40	0.67	5.95	1.10	5.42
5	10	10	22	9.80	12.00	4.50	0.61	5.90	1.12	5.29
6	10	11.3	27.6	10.90	13.56	6.28	0.68	6.40	1.12	5.72
7	10	10.7	35	7.60	12.84	7.36	0.47	7.18	1.11	6.48
8	5	8.5	44	4.00	5.10	3.54	0.25	3.47	1.07	3.25
9	7	4.5	49	0.00	3.78	2.85	0.00	3.93	1.03	3.80
Total =							**30.88**			**35.54**

$$\text{FS} = \frac{35.54}{30.88} = 1.15 \text{ which matches the assumed value.}$$

Fig.4.4b Example of analysis by Simplified Bishop Method of Slices

4.6.3.3 Translational failure analysis (For stratified soil)

Definition of terms (also see figure 4.5)

P_α = Resultant horizontal force for an active or central wedge along potential sliding surface abcde

P_β = Resultant horizontal force for a passive wedge along potential sliding surface efg

W = Total weight of soil and water in wedge above potential sliding surface

R = Result of normal and tangential forces on potential sliding surface considering friction angle of material

P_w = Resultant force due to pore water pressure on potential sliding surface calculated as

$$P_W = \frac{h_{W_i} + h_{W_{ii}}}{2} * L * \gamma_W$$

ϕ = Friction angle of layer along potential sliding surface
c = cohesion of layer along potential sliding surface
L = Length of potential sliding surface across wedge
h_w = Depth below phreatic surface at boundary of wedge
γ_w = Unit weight of water

Procedure

1) Except for central wedge where α is dictated by stratigraphy, use

$$\alpha = 45^0 + \frac{\phi}{2} \quad \text{and} \quad \beta = 45^0 - \frac{\phi}{2} \text{ for estimating failure surface}$$

2) Assume a safety factor (F_s) and solve for P_α and P_β for each wedge using the equations shown below. The same safety factor is applied to strength parameters c and ϕ. Mobilized strength parameters are therefore considered as

$$\phi_m = \text{Tan}^{-1}\left(\frac{\text{Tan}\phi}{F_s}\right) \quad \text{and} \quad C_m = \frac{C}{F_s}$$

$$P_\alpha = \left[W - C_m * L * \text{Sin}\alpha - P_w \text{Cos}\alpha\right] * \text{Tan}(\alpha - \phi_m) - \left[C_m * L * \text{Cos}\alpha - P_w \text{Sin}\alpha\right]$$

$$P_\beta = \left[W + C_m * L * \text{Sin}\beta - P_w \text{Cos}\beta\right] * \text{Tan}(\beta + \phi_m) + \left[C_m * L * \text{Cos}\beta + P_w \text{Sin}\beta\right]$$

3) For equilibrium $\sum P_\alpha$ should be equal to $\sum P_\beta$. By trial and error, find the value of Fs for which $\sum P_\alpha = \sum P_\beta$. Repeat step 2 for different values of F_s. An alternative way is to calculate, $\sum P_\alpha$ and $\sum P_\beta$ for different value of F_s and plot these quantities against F_s. Now the point of intersection of '$\sum P_\alpha$ vs F_s' and '$\sum P_\beta$ vs F_s' gives the correct factor of safety.

4) Step1 to 3 should be repeated with several potential sliding surface and the minimum F_s for all such surfaces should be taken as the F_s for the slope.

Note : (i) Depending on stratigraphy and soil strength the central wedge may act to maintain or upset equilibrium.
(ii) For ϕ=0 equations for P_α and P_β reduces to

$$P_\alpha = W * \text{Tan}\alpha - \frac{C_m * L}{\text{Cos}\alpha} \qquad P_\beta = W * \text{Tan}\beta + \frac{C_m * L}{\text{Cos}\beta}$$

An example of Translational failure analysis is shown in Figs. 4.6 and 4.7.

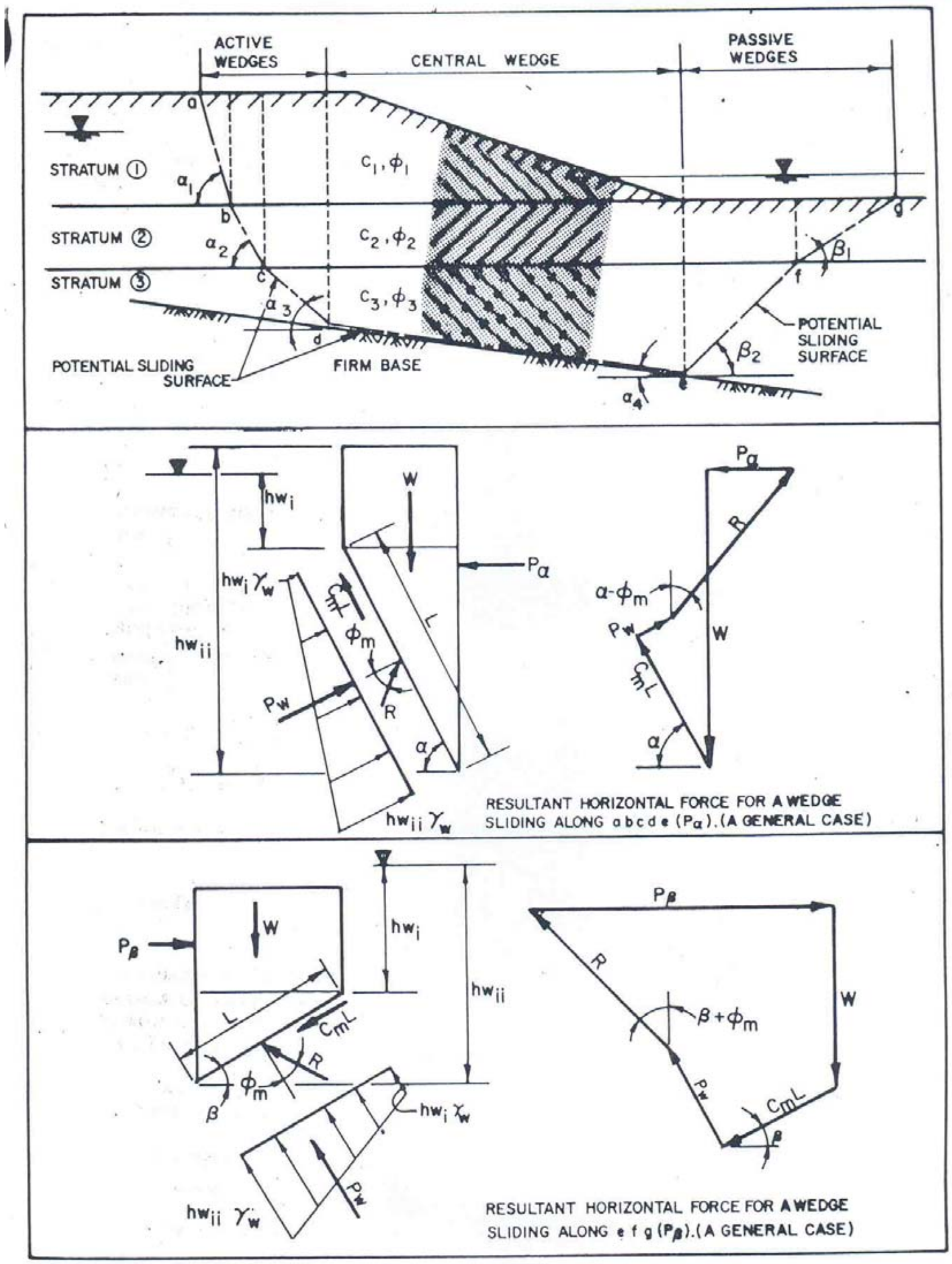


Fig.4.5 Stability analysis for Translational failure

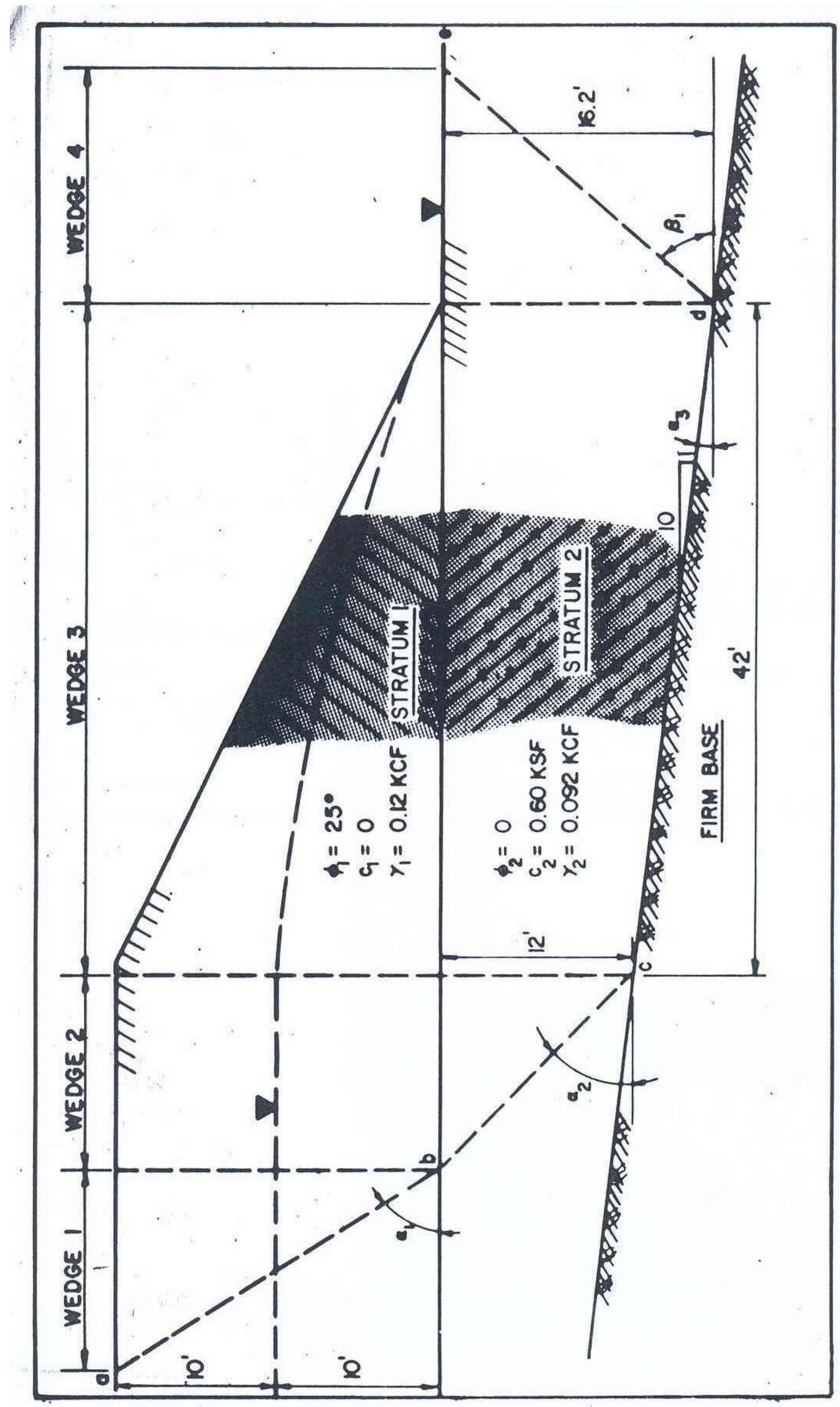


Fig.4.6 Example of stability analysis by translational failure (NAVFAC design manual 7.1)

Example of stability analysis by translational failure (continued)

Angles of wedge boundary with horizontal

Wedge	ϕ, deg	α, deg	β, deg
col.1	col.2	col.3	col.4
1	25	57.5	
2	0	45	
3	0	5.7	
4	0		45

Wedge 1 and 2 $\alpha_1 \text{ or } \alpha_2 = 45^0 + \dfrac{\phi}{2}$

Wedge 3 $\alpha_3 = \text{Tan}^{-1}\left(\dfrac{16.2-12}{42}\right) = 5.7^0$

Wedge 4 $\beta_1 = 45^0 - \dfrac{\phi}{2}$

Calculation of weights of wedges

Wedge		Length of left boundary (ft)	Length of right boundary (ft)	Width, b (ft)	Unit wt. of soil, γ (psf)	col6*col7*col8*col9 (kip)	W (kip)
		col.6	col.7	col.8	col.9	col.10	col.11
1	In Layer-1	0.0	20.0	12.7	120	15.24	15.24
2	In Layer-1	20.0	20.0	12.0	120	28.80	35.42
	In layer-2	0.0	12.0	12.0	92	6.62	
3	In Layer-1	20.0	0.0	42.0	120	50.40	104.88
	In layer-2	12.0	16.2	42.0	92	54.48	
4	In layer-2	16.2	0.0	16.2	92	12.07	12.07

Calculation of force due to pore pressure γ_w (psf) = 62.4

Wedge	h_{wi} (ft)	h_{wii} (ft)	Vertical height for calculation of L (ft)	Angle of wedge boundary with horizontal, α or β (deg)	L (ft)	P_w (kip)
	col.12	col.13	col.14	col.15	col.16	col.17
1	0	10	10	57.5	11.9	3.70
2	10	22	12	45	17.0	16.94
3	22	16.2	4.2	5.7	42.3	50.40
4	16.2	0	16.2	45	22.9	11.58

Fs (assumed) = 1.1

Wedge	Sliding surface	c (kip)	ϕ (deg)	c_m (psf)	ϕ_m (deg)	W (kip)	L (ft)	P_w (kip)	α (deg)	β (deg)	P_α	P_β
		from col.18	from col.19			from col.11	from col.16	from col.17	from col.3	from col.4	col.20	col.21
1	ab	0	25	0.0	23.0	15.24	11.9	3.70	57.5		12.24	
2	bc	600	0	545.5	0.0	35.42	17.0	16.94	45.0		22.33	
3	cd	600	0	545.5	0.0	104.88	42.3	50.40	5.7		-12.71	
4	de	600	0	545.5	0.0	12.07	22.9	11.58		45.00		29.74
										Σ =	21.86	29.74

Fig.4.7 Example of stability analysis by Translational failure

Example of stability analysis by translational failure (continued)

Fs	ΣP_α	ΣP_β
1.0	17.56	31.51
1.1	21.86	29.74
1.2	25.49	28.72
1.3	28.59	27.03

From graph

Fs =1.27

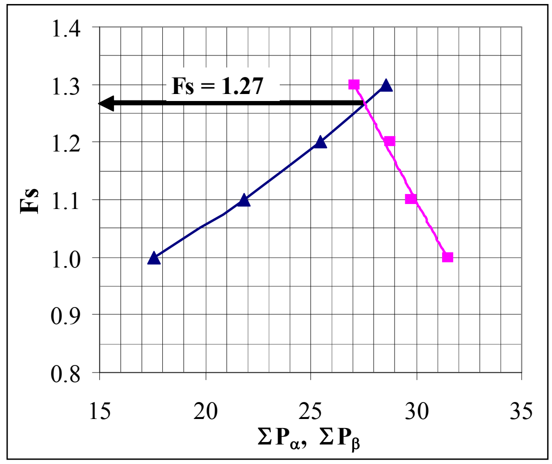


Fig.4.7 (continued) Example of stability analysis by translational failure

4.6.4 Safety Factor for Slope Stability Analysis

The numerical analysis of slope stability should ensure an appropriate safety factor, defining the ratio of mobilized over ultimate shear stress. The safety factor is, among other things, a function of:

- Method of analysis
- Variation in the design parameters, shear strength and density
- Types and frequency of various loading types (self weight, ground water, surcharge on banks and earthquake loading)
- Slope deformation and maintenance criteria, the former referring to a predominantly elastic behaviour under permanent loading and the latter too a loading condition where some plastic deformation can be accepted.

Taking into account the different causes of slope instability in the river banks, considerable experience and judgment must be employed in performing any slope stability analysis. The shear strength must be adequately defined. The loading cases assumed must be typical of field conditions. Groundwater and its effects (e.g. rapid drawdown) must be carefully considered. Finally, the method of analysis and the assumed failure surface must be consistent with the slope geometry and the soil stratification, for which the required factor of safety must be consistent with the structure's intended usage. It should be remembered that the calculated factor of safety may bear no meaning if due attention is not paid to the factors mentioned above.

For normal loading conditions a safety factor of no less than 1.5 should be ensured. For temporary loading such as earthquake safety factor of 1.25 may be accepted.

In conclusion, design criteria for river bank protection system must account not only for hydraulic aspects, but also for geotechnical factors.

4.7 Micro stability

The micro stability concerns the protection elements on the slope. These can be subject to forces from inside the revetment or external forces. The micro stability is influenced by the water level difference (gradient through the revetment) or by current forces.

During receding water level in the river, the direction of the seepage force will be directed outwardly. The ground water table will lag behind the falling river level and depending on the permeability of the soil, ground water might escape at the slope surface. The resulting uplift forces might reduce effective stresses and consequently reduce stability. Total reduction of effective stresses may therefore result in dislodging of particles, causing local instability and removal of material (Van de Hoeven, 2002).

Micro instability can be seen as a limit factor of macro instability (a very superficial slip circle) and can be determined with simple calculation rules.

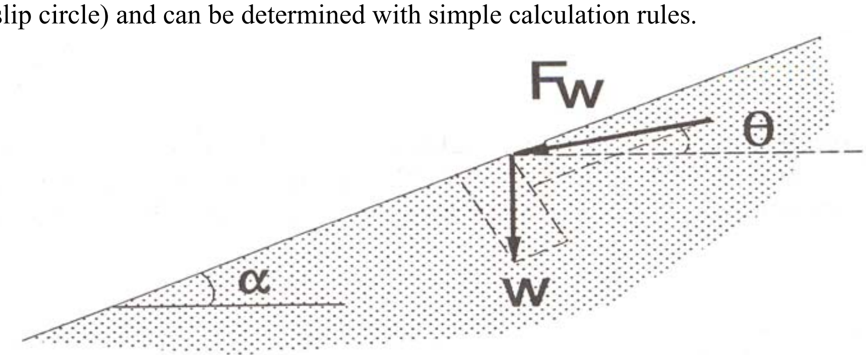


Fig. 4.9 Flow and equilibrium on slope (Pilarczyk, 1998)

Porous flow force on a unit volume, $F_w = \rho_w * g * i$ [kN]
Component of force along the slope = $F_w * \cos(\alpha - \theta)$ [kN]
Component of force perpendicular to the slope = $F_w * \sin(\alpha - \theta)$ [kN]
Component of Gravity force along the slope = $(\rho_{SAT} - \rho_w) * g * \sin\alpha$ [kN]
Component of gravity force perpendicular to the slope = $(\rho_{SAT} - \rho_w) * g * \cos\alpha$ [kN]
Specific mass of saturated soil = ρ_{SAT} [kg/m^3]

From equilibrium and using $\rho_{SAT} \approx 2000$ kg/m^3 :

$$\tan\phi \geq \frac{\sin\alpha + i\cos(\alpha - \theta)}{\cos\alpha - i\sin(\alpha - \theta)}$$

where θ = angle between the porous flow direction and the horizontal
α = angle of slope inclination.
ϕ = angle of internal friction of soil

values of i may be taken as -

0 (zero)	:	no porous flow, dry or wet slope
$\tan\alpha$	:	horizontal seepage
$\sin\alpha$	:	seppage parallel to slope

4.8 Liquefaction

The phenomenon of liquefaction is generally associated with cohesionless soil. Two necessary conditions for liquefaction to occur are the presence of loose cohesionless soil mass and a state of full or near full saturation. When subjected to cyclic shear stress (such as that induced by earthquake or wave action) such a soil skeleton tends to decrease in volume i.e. attain a relatively denser state. If the frequency of the cyclic loading and the drainage condition (even in a homogeneous deposit, a soil element at a depth will have restricted drainage compared to an element near surface) of the soil mass are such that, pore water cannot escape, excess pore pressure accumulates resulting in reduction in effective stress. A sudden loading may cause a rapid decrease (such as in less than a second) of shear strength or an insistent cyclic loading may cause a slow build up of pore pressure with similar consequences. At an ultimate state the shear resistance i.e. shear strength of the soil vanishes. The soil then behaves like a thick fluid and is said to have 'liquefied'. Because the macro stability of a bank slope is directly related to its shear strength, liquefaction poses a serious threat where such susceptible deposit exists.

4.8.1 Liquefaction potential

Not all granular soils are prone to liquefaction. As a general rule of thumb, cohesionless deposits with corrected SPT values $(N_1)_{60}$ >30 may be considered of sufficient density to pose little liquefaction risk (Earth quake engineering handbook, Wai-Fah Chen, CRC press, 2003).

The potential for liquefaction may be assessed from SPT values with the aid of liquefaction chart (Fig.4.9) developed by Seed et al. based on observations whether liquefaction did or did not occur at specific sites during numerous past earthquakes. The Cyclic Resistance Ratio (CRR) curves on this chart represent limiting conditions that determine whether liquefaction will occur or not. The intensity of shaking is expressed in terms of the average effective Cyclic Stress Ratio (CSR) defined as

$$\mathrm{CSR} = \frac{\tau_{av}}{\sigma_{v0'}} = 0.65\,\frac{a_{max}}{g}\,\frac{\sigma_{v0}}{\sigma'_{v0}}\,r_d$$

where τ_{av} = equivalent shear stress amplitude

a_{max} = peak horizontal acceleration at ground surface generated by earthquake

g = acceleration due to gravity

The term $\frac{a_{max}}{g}$ can be taken as the seismic zone coefficient, Z (Fig.4.2)

σ_{v0} = total vertical stress at depth z

and σ'_{v0} = effective vertical stress at depth z

r_d is a stress reduction coefficient which may be approximated as follows :

$$r_d = 1.000 - 0.00765z \qquad z \le 9.15\,\mathrm{m}$$

$$r_d = 1.174 - 0.02670z \qquad 9.15\mathrm{m} \le z \le 23\,\mathrm{m}$$

$$r_d = 0.744 - 0.00800z \qquad 23\,\mathrm{m} \le z \le 30\,\mathrm{m}$$

$$r_d = 0.500 \qquad z > 30\,\mathrm{m}$$

Combination of corrected blow count $(N_1)_{60}$ and CSR that plot above the appropriate CRR curve indicate a high probability of liquefaction, whereas those plotting below suggest a low probability of liquefaction. In the chart (Fig.4.9) individual CRR curves are shown for various amounts of fines (percentage of material smaller than 0.075 mm i.e. passing #200 sieve). The CRR curves in Fig.4.9 are for an earthquake magnitude of 7.5. For earthquake of a different magnitude, a Magnitude Scaling Factor (MSF) should be used to shift the CRR base curve vertically according to:

$$CRR_{corrected} = CRR_{7.5} \times MSF$$

The range of MSF values recommended by the NCEER committee is provided in Fig.4.10.

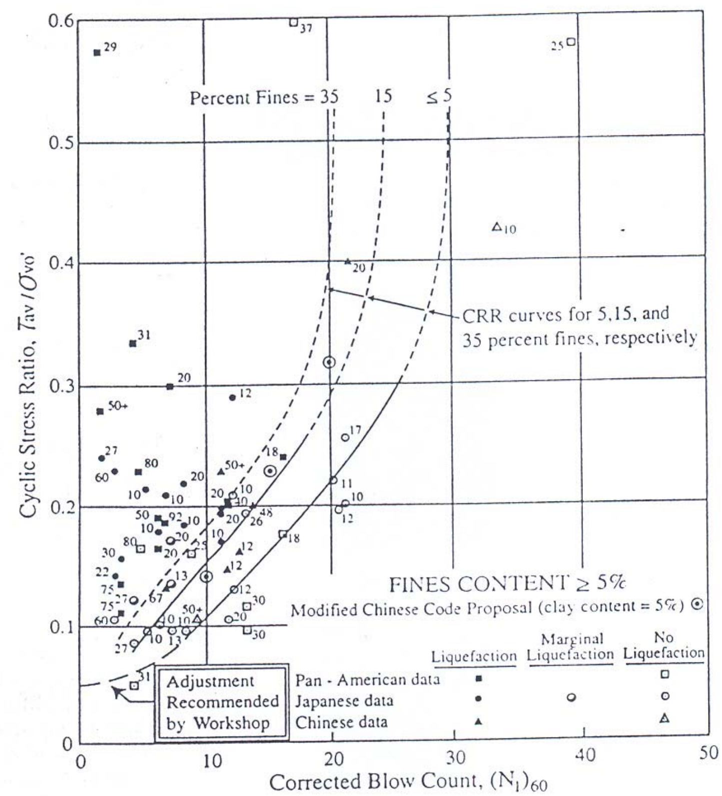


Fig.4.9 Liquefaction resistance based on SPT (Earth quake engineering handbook, Wai-Fah Chen, CRC press, 2003)

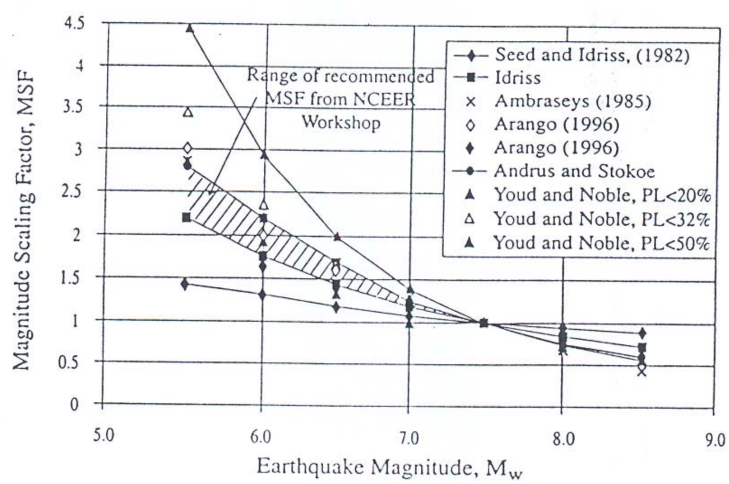


Fig.4.10 Range of MSF values (Earth quake engineering handbook, Wai-Fah Chen, CRC press, 2003)

The corrected SPT $(N_1)_{60}$ is obtained from the field SPT value N_f by applying a set of correction factors to correspond to 60% of the maximum potential free fall energy of a safety hammer operated by rope and pulley. Field SPT tests that are done in accordance with ASTM D1686 should yield consistent energy ratios.

$$(N_1)_{60} = C_N N_f C_E C_B C_R C_S$$

where C_N = correction factor for overburden. Through this correction factor the SPT value is adjusted to a standard stress of 95.6 kN/m^2 (1 ton/ft^2)
C_E = correction factor for energy
C_B =correction factor for borehole diameter
C_R = correction factor for rod length
C_S = correction factor for sampling method

Table 4.3 SPT corrections (Table 7.5 of Earth quake engineering handbook, Wai-Fah Chen, CRC press, 2003)

Correction factors	**Variable**	**Term**	**Value**
Energy ratio	Donut hammer	C_E	0.5-1.0
	Safety hammer		0.7-1.2
	Automatic trip donut type hammer		0.9-1.3
Borehole diameter	65-115 mm	C_B	1.0
	150 mm		1.05
	200 mm		1.15
Rod length	3-4 m	C_R	0.75
	4-6 m		0.85
	6-10 m		0.95
	10-30 m		1.0
	>30 m		<1.0
Sampling method	Standard sampler	C_S	1.0
	Sample without liner		1.1-1.3

The oveburden correction factor C_N can be approximated by

$$C_N = 0.77 \log_{10} \frac{20}{\bar{p}} \qquad \text{valid for } \bar{p} \geq 0.25 \text{ ton/sft}$$

where $\bar{p}$ is the effective vertical overburden pressure in tons/sft at the elevation of the penetration test.

4.9 Filters

A filter layer is usually referred to as the layer between the top (or armour) layer and the base material. In case of a falling apron, the top layer/protection element is directly placed on the bed material. It has to be heavy enough to be stable against the current and also it has to retain the soil. Incipient motion starts at 0.25 m/s with a d50 of about 0.1-0.3 mm.(Oberhagemann et al., 2006).(
Fig. 4.11).

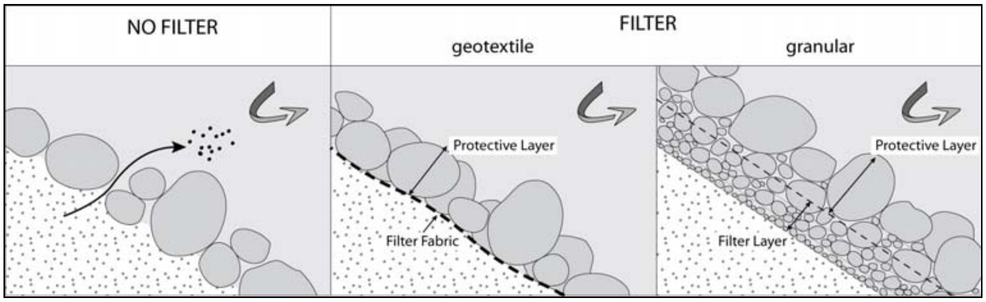


Fig. 4.12 The principle of filters (Oberhagemann et al., 2006

The criteria for granular filters concern mainly the grain sizes, grading of filter to subsoil, the strength and weathering of the stone material and the unit weight. Requirements for grain sizes in relation to filter purposes are based on the general filter criteria.

It might be necessary to use more than one granular layer in order to achieve the required filter characteristic. In that case 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. (Fig. 4.12)

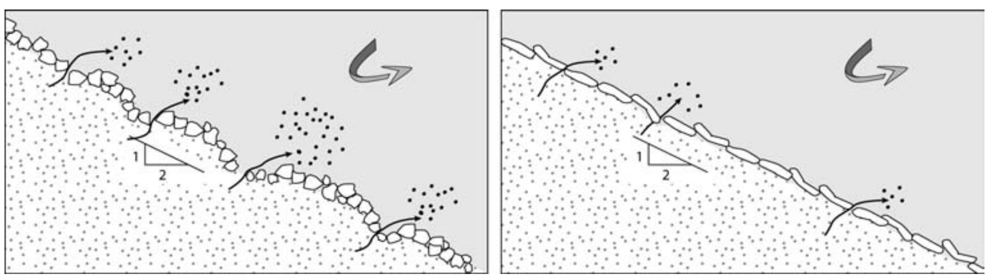


Fig. 4.13 Falling aprons provide about one layer of slope coverage after launching, which leads directly to erosion of the underlying soil (Oberhagemann et al., 2006)

Traditional design formulas for Falling Aprons aim to provide a cover thickness of 1.5 times the average thickness of the individual protective elements after launching. This is the maximum thickness usually achievable under field conditions, as the launching elements slide easily over the subsoil but not over each other. This may lead to erosion of subsoil below the armour layer (Fig.4.12). Therefore the stability of Falling Aprons depends on two alternatives, either having filter material to separate the layer of protective elements from the subsoil, or placing multiple layers of protective elements. (Fig.4.13).

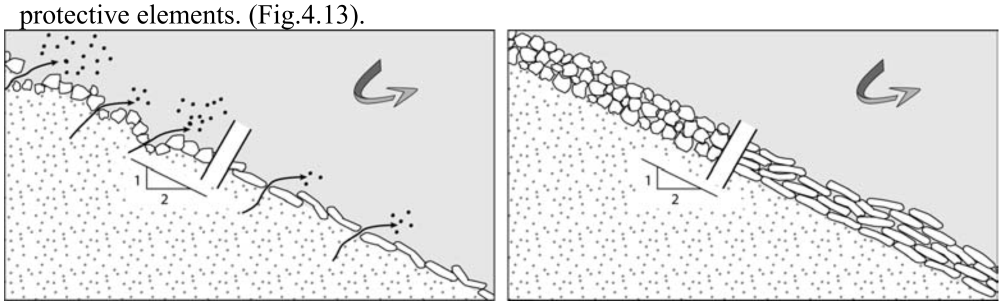


Fig. 4.14 Multiple layers of rocks and geobags are capable to protect the fine subsoil

4.10 Soil Characteristics of the Jamuna River/ Meghna River

The floodplain of the three main rivers Jamuna, Padma and Meghna consists 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 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 Jamuna. (FAP 21/22, 2001a).

The main river sediment of Jamuna consists of poorly graded fine sand of loose to medium density. Major part of the river bank is sand and silt. It comes from the Himalayan Mountains and consists of mineral quartz, rock fragments, heavy minerals and mica. The amount of mica varies depending on the author. Gales e.g. documented 50%, others a bit less, while a specific analysis by the Bangladesh University of Engineering and Technology (BUET) shows a percentage of 7 % to 11% (Fedinger, 2006).

In the consolidated soil commonly found along river banks, slopes of 1:2 are at the borderline of stability. The angle of internal friction φ, which represents the shear strength of sand, is in this case:

$$\varphi \sim 28 - 30\,^{o}$$

$$c \sim 2\text{-}4 \text{ kN/m}^2$$

A considerable amount of slopes, however, have inclinations of 1:1.5 or even slightly steeper. This results in an effective angle of internal friction:

$$\varphi \sim 32 - 35\,^{o}$$

In unconsolidated char soil only flattened slopes of 1:3.5 or less are stable (Fig.4.14). Due to the less consolidated material the angle of internal friction is lower then that of of consolidated material. This has a significant influence on the slope angle of the launched apron.

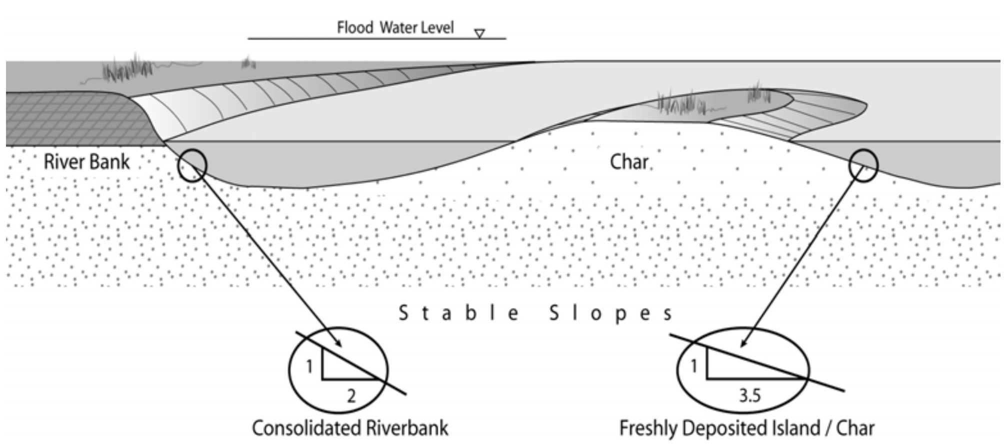


Fig. 4.15 Stable natural slopes along consolidated riverbanks and recently deposited chars

In some areas the river bank stands nearly perpendicularly. In this state the sand is only able to stay vertically if there is cohesion, either due to clay or due to the apparent cohesion (capillary forces). This apparent cohesion vanishes when the sand is drying or when it is saturated to a degree of more than 70% (Fedinger, 2006).

In the estuaries where a river is meandering or frequently forming new channels it is very often found that the type of sediment deposition is widely varying even over short distances. In such areas, for the design of construction measures a generalised soil profile and a generalised riverbank have to be considered. For this reason, for comparison and estimation stability of several cross-sections may be investigated.

4.10.1 Flow slides

Miscellaneous particles (such as mica), which have a plate-like shape, have an enormous influence on the behaviour of the sand. The presence of it within sand increases its void ratio (porosity), modifies its volume change characteristics and introduces a collapse potential, particularly when under low stress and when sheared in extension. Shear testing with different mica percentages have shown that the angle of repose increases the higher the mica percentages are in the sand (van de Hoeven, 2002).

When the process of sedimentation is not disturbed, the mica minerals cover the sand grains in a thin layer. The risk of flow slide due to mica increases when the mica minerals form a texture (due to different settling velocity), in which a thin and impermeable or nearly impermeable skin of mica flakes encloses the permeable but saturated granular particles (Fedinger, 2006).

Below the water table the sand is saturated and flow slides may be observed in places due to quick change of stress level in the area of the toe. Change of stress level can occur due to:

- Scouring: scour occurs in a very short time, balance of pore water pressure cannot be achieved due to short period (FAP 21 considered a velocity of formation of scour of 0.2 – 0.4 m/day to be dangerous for Jamuna soil. In Jamuna River the formation of scour has been: 5 m per day, 12 m in 10 days and 20 m in less than one month).
- Hydraulic dredging: If executed too fast and undercutting the slopes, it will produce the same effects of scouring. Therefore dredging has to be conducted at a very slow rate, excavation in front of toe shall be less than 0.2 m/day.

Flow slides normally start in the area of the toe of slopes and continue in a retrogressive manner. Flow slides and erosion have nothing to do with the mechanism of geo-mechanical slope failure.

4.11 Considered Slope

Geotechnical considerations for bank protection should not primarily start with the soil characteristics for determination of stable slopes for existing river banks. In case of design/construction of riverbank protection the existing slopes of the riverbanks shall have to be taken into account. For the examination of slope stability of the riverbanks a generalized cross section of the riverbank is needed.

For determination of the most unfavourable slope, cross sections of a certain river reach have to be considered individually, as only this allows identifying areas that deviate from general assumptions. The most unfavourable cross section at the brink of geotechnical failure for the project may be selected in this way.

From cross-sections of Jamuna and Padma River in PIRDP and MDIP area it was observed that approximately 50% of the slopes have an average inclination of about 1V:2H to 1V:2.5H. On this basis, a representative cross section has been worked out, which corresponds to frequent and unfavourable situations. There are also slope inclinations flatter than 1V:2H, which may have been formed by deposition, but also by reduced soil characteristics and a more complex stratification. The steep slopes are to be ascribed to "hidden safeties", such as higher cohesion in clay and cementation of sand.

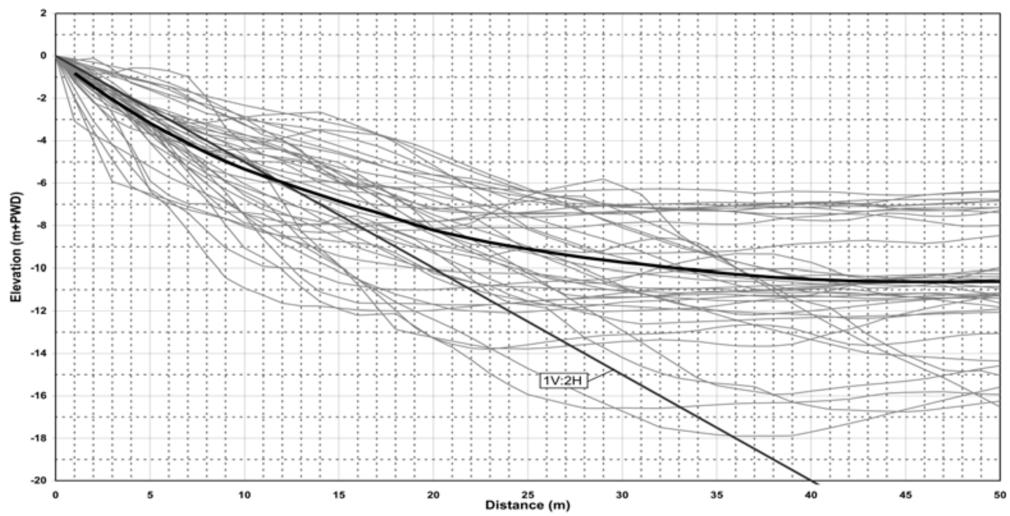


Fig. 4.16 Measured cross sections at the PIRDP in 2001

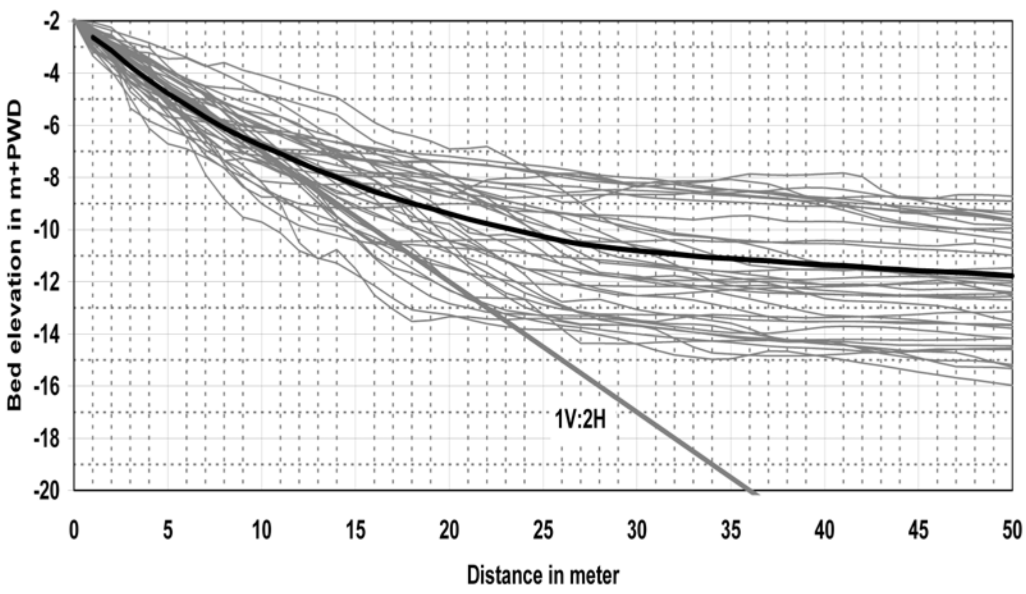


Fig. 4.17 Measured cross sections at the MDIP in 2001

After evaluation of cross-sections the following slopes of the river bank may be considered to be representative for steep observed slopes and subsequent calculations of slope stability. The central part of the considered riverbank has a 1V:1.25H slope and the lower part a 1V:1.5H slope.

The model tests conducted in Vancouver, Canada for incipient motion on standard protection element (250 mm rock or 300 mm cc block or 126 kg geobag) showed an average flow velocity of 2.6 m/s for an inclination of 1V:1.5H. This flow velocity will not generally be encountered neither in the Lower Meghna River nor in the Jamuna River. Therefore geotechnical considerations are also covered by this part of the model test. A horizontal berm (minimum 2.0 m) below lowest water level and addition of a flatter slope of 1V:2.5H (Fig.4.17) from the berm to floodplain level adds to the stability of existing section up to required extent.

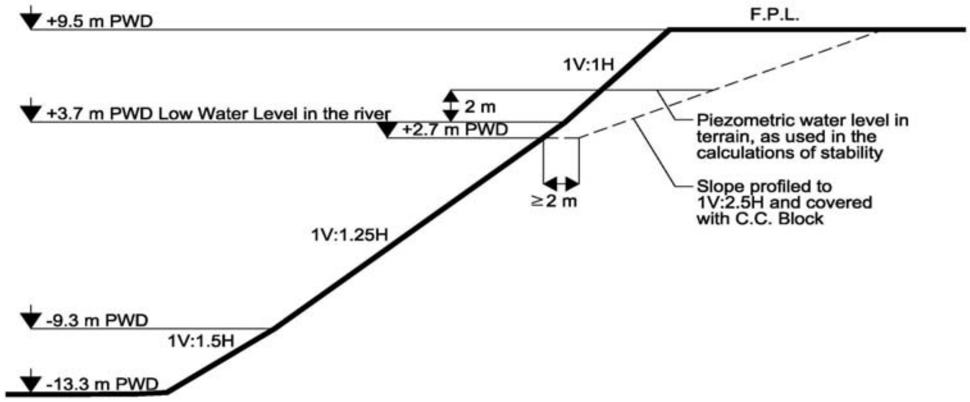


Fig. 4.17 Representative steep cross section, levels for PIRDP

CHAPTER FIVE

General Planning of Riverbank Erosion

5.1 Riverbank Erosion Management

The complex, interrelated activities required for the development and successful implementation of planning for river training, design, construction and maintenance call for a multi-disciplinary approach. The ***best overall results are obtained from a total concept of river training*** where all aspects are handled by a single integrated team working in partnership with stakeholders.

Along all the major rivers in the country, much of the damage to human settlements and infrastructure has been caused primarily by riverbank erosion and channel instability, not from flooding. Since 1973, the Jamuna River has progressively widened eroding over 70,000 ha of flood plain land while accreting only about 11,000 ha (CEGIS 1997). Like Jamuna all other major rivers are also expanding. Expansion is taking place through destruction of flood plain land (homesteads, towns, growth centers) and creation of short live char-land.

People may be able to adapt to recurrent flooding, but they find riverbank erosion and channel instability difficult to cope with. Social impacts on riverbank erosion include increased impoverishment and landlessness, loss of income generation, migration of displaced population reduced security and social displacement.

Riverbank erosion causes disaster for many families every year. The instability of major rivers makes all efforts to protect the population and infrastructure difficult and expensive. Sustainable and reliable riverbank protection work is a key element in providing safe and stable living conditions along the densely populated banks of major Rivers in Bangladesh. The inherent river instability of all major rivers in Bangladesh can only be mitigated through the stabilization of longer reaches. As bank protective works are capital intensive, there is strong quest for economically feasible and sustainable bank protection methods.

Sustainable bank protection reduces the risk of disaster- the loss of lands and infrastructure and provides stable living conditions. It is desirable that persons responsible for design should be closely in touch with construction, inspection and maintenance, so that the implications of any developments and occurrences are understood. Unless there is feedback to design and construction from maintenance experience, progress in updating practices will lag. In this report a systematic approach on REMS need to be developed which will play a vital role for erosion management in Bangladesh.

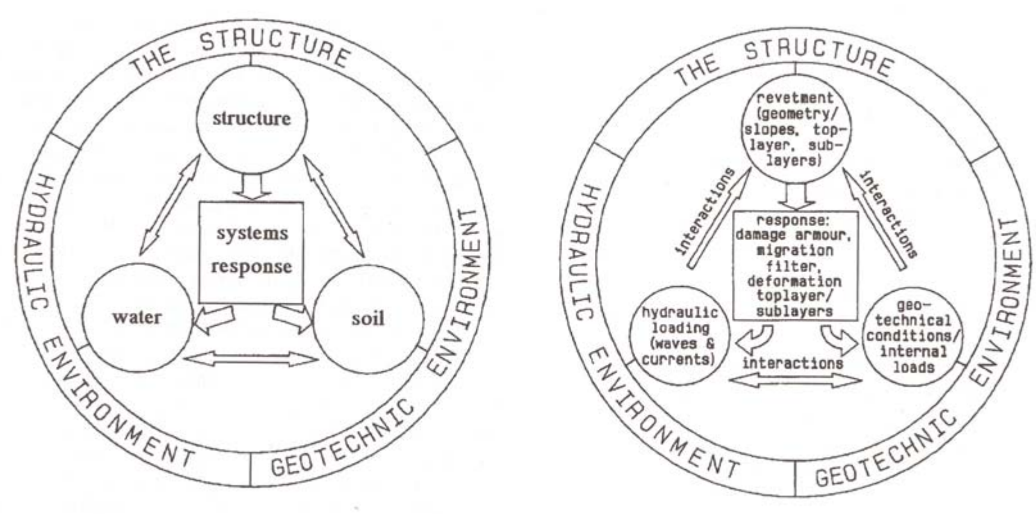
Riverbank Erosion Management (REMS) may be conceptualized as an integrated system that takes a holistic view of all issues related to riverbank erosion management. A systems approach to riverbank erosion management would bring about rationality in design, execution and management of erosion control measures replacing the patchy and adhoc approach that currently dominates. REMS will not be satisfied with the execution of a structural intervention only but would also like to ensure its sustainability in terms of maximization of benefits to concerned people, its proper operation and maintenance and its impact on the environment.

River stabilization distinguishes between river training works, covering the whole width of the river and bank protection work mitigating bank erosion and maintaining existing flood plains often at one riverbank.

Planning of riverbank protection needs to assure main aspects like availability of funds and knowledge about the river conditions. These elements are combined in the Riverbank Erosion Management (REM). REM include river monitoring, decision making tools, standard designs, development of prediction tools, prioritization of interventions, and quality assurance during implementation, and monitoring for sustainable and reliable bank protection works.

Riverbank Erosion Management (REM) is a combination of structural and non-structural aspects, such as erosion forecast, development of cost effective riverbank protection techniques, improvement of construction and supervision technique during dry season and during emergency, development of standard bank protection design for major rivers to be implemented during emergency, development of monitoring system, making provision of block allocation of funds, introducing flexibility in selection of final protection area and extent of protection, and incorporating longer term of contracts and stakeholder participation.

5.2 Design Principles



general approach

protection systems

Fig. 5.1 Principles of integrated design (Pilarczyk, 2000)

It is very common in Bangladesh to design riverbank protection focusing mainly on the hydraulic loads. This is insufficient for a stable structure. It should be standard to consider the interaction of the protective elements, hydrodynamic loads and geotechnical conditions (Fig.5.1). Only taking by account of all these three elements, a structure can endure its loads for a reasonable time (Pilarczyk, 2000).

5.2.1 Phased Implementation Concept

Bangladesh finds it difficult to protect long reaches of its eroding bankline using sustainable bank protection structures which are very costly. The need is therefore to identify alternative concept and materials for erosion mitigation that would prove cost effective even for protecting agricultural land. This need can be met by using an 'Adaptive Approach' that stabilizes the riverbank along an alignment natural to the river system. The riverbank erosion management in Bangladesh should be addressed in a way that assures the proper measure in proper time. An adaptive approach, which means that interventions are planned according to observed and predicted river behavior. An adaptive approach for a riverbank stabilization, which comprises a revetment along the bank incorporating a launching apron of loose material, would relate to the following aspects:

- To provide protection initially to the most vulnerable reach and make provision, including design concept and fund allocation, for taking care of any sudden increase in erosion attack in the adjacent reaches of the river in the subsequent time; this would allow an opportunity to limit the initial capital investment and use the additional provision only when there is a need;
- To place apron material sufficient to launch to the minimum anticipated scour, and then replenish the apron material in the subsequent phases, as and when needed.
- Storing appropriate materials for emergency protection;
- Regular monitoring during monsoon;
- Immediate intervention when failure occurs.

The variable river environment requires a flexible approach, commitment for continuous monitoring, and interventions over years after construction.

5.2.2 Erosion Risk and Duration

Provision of protection along a major river that is subjected to large scale erosion would have to deal with uncertainties relating to design parameters, and consequently there would be risks of failure. Bank protection works can be considered adequately designed and built if the probability of failure of the structure is sufficiently low and the various component of the structure are attuned to each other in such a way that a localized damage to one of the components does not lead to the failure of the structure as a whole. Generally, higher is the initial investment, lower would be the risk of failure (and lower would be the maintenance/repair cost) and vice-versa. The aim should be to contain the risk of failure to an acceptable level, while not wasting scarce resource by over caution.

To analyze the risk of failure of the structure, a good appreciation of the failure mode of each structural component must be known. There would always be inherent risks in the design and construction of works to control large, dynamic river systems. These risks can be managed by using proven design criteria and/or by making provision for carrying out controlled remedial/rehabilitation works.

Type of risks anticipated are Morphological Risk that includes (i) Upstream bend development, (ii) Downstream bend development, (iii) Severe channel attack and also

Structural Risk like (a) Improper launching of apron material, (b) Slope failure, (c) abnormally high scour etc.

When humans begin to use the river's resources, they must remember that the as-built newly constructed channel geometry must be designed so that it can accommodate all magnitudes of future flows with the least amount of maintenance. Any man-induced improvements that alter the channels water load and/or its sediment load will initiate geometric changes, which may invalidate the original design. The river dynamics must be considered when designing works to stabilize short reaches of the river in an attempt to design against conditions that might adversely affect the structure. An adverse condition that could occur would be the development of a relatively large channel that directly impinges on the structure. A review of the existing conditions at Jamuna shows that an angle of attack of about 45° is as severe as would be expected if one considers the Jamuna as a whole. Another adverse condition that might occur would be that episodic erosion would develop downstream from the proposed stabilization works that would threaten the protection work.

To address all these potential risks continuous monitoring and observation shall be done and provision should be made to address all these probable risks. To address the potential problems of embayment upstream or downstream provision for extension of revetment work in phases shall be kept. In case of severe attack causing deeper scour than anticipated, additional material to be added to reinforce the area under attack. An effective Operation Maintenance (O &M) with adequate provision of fund and materials can also take care of most of these problems.

It is observed that planform of the river Jamuna displays elements of braiding, meandering and anastomosing when viewed at different scales of space and time. This fact is crucial to understanding and explaining the short, medium and long term morphological development of the river. The Jamuna River has been increasing its width considerably over the last three decades. Data received from CEGIS clearly shows that during the period from 1973-2000 the area of Jamuna River within the bank lines increased by about 41%, the increase in river width during this period is 122m/year. Recent studies have shown a lesser increase of width (CEGIS, 2008).

On average, erosion would last a couple of years, then there would be no erosion for a few years (may be accretion), and after another few years erosion would come back. In the short to medium term, the process of bank erosion cannot be regarded as an entirely stochastic process. Erosion at a given location will depend strongly on the morphological history and on the erosion at other locations (time, space dependency). Analysis of erosion records along the right bank of Jamuna shows that erosion some times does have the tendency to migrate downstream for some time.

Erosion will slow or cease when the embayment is abandoned due to either (i) abandonment of the near bank channel at the bifurcation upstream, or (ii) cut-off of the embayment by a chute dissecting the braid bar.

It is difficult to put a timescale on the sequence of embayment formation and abandonment, but experience from FAP-1 suggests that this cycle is seldom shorter than 4 years or longer than 10 years.

At all locations of Jamuna River very fast morphological changes occurred, both siltation as well as scouring. At Bahadurabad maximum observed scour was 6.0 meters in ten days, 8.0 meters in three and half weeks and 12.0 meters in one month. Scour at Sirajganj reached 20.0 meters in three and a half weeks, with a peak of 5.3 meters in one day during 1998 flood. Highest measured scour at Kalitala during 2001 flood was 5.0 meters in three days and at Mathurapara hard point 12.0 meters in 10 days in September 2000.

Siltation of 11.0 meters in 24 hours occurred in Sirajganj during construction of Sirajganj Town Protection in 1997. Maximum siltation at Bahadurabad reached at 20.0 meters in three and a half weeks during 1998 flood. At Kalitala nearly 25.0 meters siltation took place from July 2000 to February 2001, and more than 30.0 meters scour occurred from June to September 2001. It is observed that both high erosion and siltation rates can occur during moderate floods at similar stages.

The Jamuna River is continuously widening from an average 8.5 km in 1973 to presently nearly 11.5 km with an average rate of 122 m/year. Widening has slowed down since 1992, with an average rate of only 67 m/year. During earlier high floods the rate of widening persisted for several years. After the 1998 record flood it came down to the average already one year later. The present system seems to be less sensitive now to the impact of a major event. This could be a first indicator of a general slow down of widening, assuming 12 km active channel width as a possible mean equilibrium width.

The reach of Jamuna River between outfall of Hurasagar and the Jamuna Bridge is significantly narrower than other reaches. The river area within the banklines of this reach increased by 41% in 2000 compared to the area of 1973. In this period the water area increased by 30%, showing that the river gets wider and shallower. The difference from the average is contributed to above average char and bar area growth. In 2000 the water area was 5% less than 1989 after the two record floods in 1987 and 1988

General information about bank erosion of Jamuna River is:

- High threat of more than two hundred meters erosion per year occurred on an average. The influence of locally different soil still needs to be assessed.
- The reach of high erosion was at maximum 6 kilometers and on an average 3.5 kilometers long.
- Periods of erosion last longer than periods of accretion resulting in net loss of river bank.

The left bank of the Ganges river is less vulnerable to erosion than right bank. In some reaches erosion rate as high as 1000 m/yr was found to occur, while in other reaches the erosion rate was about 10 m/yr. Due to presence of Pleistocene clay deposits those appear to a number of natural hard points which limits erosion rate at those locations, while at other points erosion occurs at different rates (Hossain, 2006). The channel development and abandonment in Padma River is not as frequent as in Jamuna River. But the declining of the anabranches in favor of its counterpart is a very common process. Channel abandonment can only be predicted if sedimentary features are found to be present within 5 km upstream of the bifurcation. Almost 81% of the bifurcations are found to migrate downstream, which is very close to that of the Jamuna River. During the last four decades the Padma River has changed its planform from braided to straight and straight to braided again (JICA 2004). Starting from a combination of meandering and braided planform in

1967 the river appeared as a straight river in 1980. By 1993, the planform of the river appeared similar to that of 1967. CEGIS (2005) analyzed the planform data extracted from aerial photographs and satellite images. According to this study the river has changed its width, braiding index and amount of erosion and accretion along with the changes of the planform. In 1967 the length-averaged width of the river was 7.7 km and it was reduced to 6.7 km when the river was straight in 1980. The river has attained its maximum length-averaged width of 10.3 km in 2005.

5.3 Planning and Design

5.3.1 Erosion and Morphology Prediction

Erosion predictions and their proper dissemination to national and local organizations as well as to communities have a great potential to enhance water resources planning both at macro and micro levels, reducing national losses and suffering of erosion vulnerable people. Predicting morphological changes provides an opportunity to concentrate protection efforts on immediately threatened reaches and reduces the risk of implementation failure in the available time of low-water construction season. The identification of the potential for the riverbank erosion, and subsequent need for stabilization, is best accomplished through observation.

Prediction of erosion and morphological changes one year ahead with different probabilities can be of much help for taking care of certain abnormal situations that may occur in next monsoon. Preparatory activities of the Government for facing the probable calamities through erosion may be of great help to overcome the difficulties in proper time and sequence. The prediction method and the tools developed by CEGIS need to be more refined and dependable through verification with the events already occurred and forecast made. The monitoring unit of BWDB should have close liaison with the planning & design unit and CEGIS or any expert group responsible for prediction of bank erosion.

5.3.2 Prioritization

In view of the limited fund available not all the places vulnerable to erosion can be protected at a time. Hence a prioritization of the protective works needs to be undertaken. Prioritization of the works needs to be done through combining the warning of erosion forecast and the pre-determined erosion criteria. The predetermined criteria may follow the guidelines like importance of the place or structure to be saved, severity of attack and consequences of not protecting the bank immediately. The predefined prioritization criteria shall be approved by the competent authority and placed for guidance of the planners and implementers.

The prioritization criteria may be:

1. Defend existing protective works
   - Measure against outflanking of an existing bank protection structure,
   - Repair of damaged bank protection structures,

2. Defend Infrastructure
   - Protection of important highway/railway links,
   - Protection of place of historical importance.

3. Protection against drastic environmental changes
   - Change of river course that may endanger stability of environmentally important places and cause environmental imbalance,

The protection criteria shall be established by a committee of experts mainly from design and planning office and approved by the competent sectors of the Government.

All the alternatives may be prioritized to achieve the most important aspects, or prioritization may be done in clusters to attain the desired goal.

5.3.3 Protection Length

Length of protection refers to the longitudinal and vertical extent of protection required to adequately protect the riverbank. Analysis of site-specific factors is necessary to define the actual extent of protection required. The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions. Fast changing river conditions make it ineffective to build short lengths of localized bank protection and require the protection of longer reaches. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material.

Bank erosion in major rivers of Bangladesh extends to several kilometers in one place or some times in different reaches depending on the flow pattern and morphological behavior. Protection measure taken on a short reach where attack on several kilometers are active or protection measure undertaken in one reach where at least another upstream reach has to be protected simultaneously may invite failure to the attempt and cause more harm to the properties and infrastructures. So criteria shall be formed during design formulation to suggest minimum length extent of protection works to be taken up in a year of a particular river.

The protection work selected as suggested through prediction tools and priority criteria should also contain a minimum length of bank protection in one reach or several reaches to attain a sustainable and effective bank protection. The length of protection work taken up in a year in consideration of priority, limited fund availability, and available construction period should be aimed at making a long protection reach. The basis for such selection shall be prediction and continuous monitoring through any expert group. The long revetment provided shall have the additional advantage of maintaining a navigation route. The vertical extent of protection shall be based on expected design scour depending on location of the planned protection measures. It may be noted that implementation of any bank protection work not addressing the minimum requirement shall be avoided.

5.3.4 Standardized Interventions

The stability of unprotected riverbanks 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 protection measure. It is stressed that the proposed protection scheme can possibly affect the overall stability of the river or a river channel.

In designing stabilization structure, it is important to consider cost and it is even more important to analyze and understand the possible failure mechanism that causes maintenance problems. The dominant criteria for the design of bank protection are scouring and undetected soil weakness. Attempt should also be made to use locally available construction materials and construction technique.

New construction methods make it possible to design and implement river works of increasing complexity and dimensions, at acceptable cost. For instance, the use of geo-textile instead of classical gravel filters, combined with powerful machinery reduces both costs and time limits for executing large scale river works.

The use of stone as launching material considering both construction and maintenance in the type of structure (revetment) is well-established and tested, but replacement of launching material with sand filled geo-bags needs more observation and monitoring.

Another criterion for design is to select appropriate launch depth. The possibility of damage to the revetment due to slip failure, caused by increased pore pressure in the soil, and/or due to gully erosion of the protected bank during high intensity rainfall or over-bank flow is another potential risk.

A designer can address the problem in two ways:

1. Either the protection is built to deepest scour level, which makes the structure expensive, or
2. The protection is built to the existing bed level including protection of a certain strip of riverbed in front, following slope stability considerations. At the same time a prediction model is set up, to predict the river behavior one year in advance, regular monitoring of the structure is carried out, and provisions to strengthen the toe are at hand.

With the trial for replacement of rocks, cc blocks or gabions as apron material by sand filled geobags below low water level, attempt should also be made to find suitable protection material above low water level. The present practice of manufacturing cc blocks for slope protection needs improvement. The specification followed by BWDB for manufacturing cc block, is a concrete block having a minimum strength of $9N/mm^2$. The elements specified for the concrete are 40 mm downgraded shingles, sand of FM≥1.5 and required quantity of cement. The concrete having the same or higher strength and unit weight of same range can be manufactured from pea gravels, coarse sand and cement in factories manufacturing the sand-cement or concrete blocks used in lieu of bricks. Since 40 mm down graded shingles are not readily available in Bangladesh in sufficient quantities, an alternate material need to be searched to have the concrete of similar strength at a reasonable price and acceptable quality.

Not only the apron material, the construction methodology for bank protection including the period of construction, equipment needed for the construction, method of dumping/placing apron material, minimum coverage to be made in a day’s work, skilled man-power, monitoring technique etc should also form the part of design package for this type of construction. Attempt should also be made to use optimum size of apron material, stable against the type of flow encountered, which can be easily handled and a better coverage is made during dumping in a day’s work.

Lower limit of laying cc blocks on slopes should be fixed in a way that placing of cc blocks on slope does not require any dewatering. In order to achieve an economic and workable solution the average low water level can be marked in the drawing, with the direction to extend the limit of hand placed blocks up to 30cm below or above that boundary as water level permits within the allowed time span. The variation of this lower limit should follow a stretch of equal reach of slope protection.

One of the most critical criteria is the gradation of launching material. The gradation available in most of the textbooks was developed with many years of prototype experimentation. The recommended graded material has various percentage of relatively fine and course particles. The fine material is thought to help launching process and provide a filter after launching. This type of gradation is not possible by use of cc blocks. Properly graded rocks can attain the required criteria, but underwater dumping of such graded material is very difficult and expensive, because the material has to be placed or dumped just at the place where it is needed. Dumping from a distance will initiate segregation (Neill 2004, Hassain 1993).

According to Halcrow, the suggested apron protection in PIRDP is 200 units of geo-bags and in MDIP 133 units of geobags. One unit geobag weighs 504 kg. (1no-126kg +1.61 nos-78kg +3.50 nos-36kg +11.45 nos-11kg). Suggestion for gradation is made in consideration of gradation curve for launching apron with hard materials like stone boulders or quarry stones.

The geo-bags being flexible enough to accommodate any shape can be limited to one or two sizes. Observation made in Vancouver Model test does not specifically support so many bag sizes. Observation made in prototype works also provides very little support in favour of bags of many different sizes.

Assumed launch thickness of 0.91m for geobag in Feasibility Study also appear to be on higher side. The reduction of thickness may be possible through close monitoring and can be covered through phased and adaptive approach in places where a more intensive protection is needed. A specialized survey unit with the modern survey equipments may be deployed for continuous monitoring of the underwater condition of the protected area and also the upstream and downstream areas of all protected stretches.

5.3.5 Planning and Funding

Riverbank protection should be taken up in a timely and strict coordinated manner using (i) Prediction tools, (ii) Observation, (iii) River surveys and under water investigations and (iv) Detail and integrated analysis of existing data.

In an analysis of any river system, it is necessary to consider not only the numerous geologic, hydrologic, and hydraulic variables but also, the local geometry which is continually attempting to respond to the ever changing hydraulic variables with the concomitant sediments and local natural controls.

All these data analyzed and refined over time along with pre-determined prioritization criteria could provide planning horizons for (a) different types of rivers and (b) different levels of details.

Budget allocation for the Annual Development Plan (ADP) shall be based on erosion prediction made during first quarter of the year on the basis of dry season satellite image. Adaptation and maintenance of existing structures shall be based on standard designs. The provision of maintenance shall be kept on the basis of standard design, expected length and depth of river. The block allocation provided for such maintenance shall be finalized after the pre-work survey conducted at the end of monsoon. Any emergency repair needed in those reaches or any other vulnerable reach already identified through prediction and observation, shall be taken care from block allocation maintained from the O&M budget.

5.3.6 Stock Piling of Construction Materials

Based on the prediction, the contingent plan for protection of a certain reach is made for the next year. Provisions have to be made also in terms of materials and equipment ready for emergency works (phased installation). There are certain areas that may need immediate attention during the flood season. To face such situation a stockpile of materials may be made at vulnerable locations to start emergency repair/protection works in a very short span of time. Stockpiled materials and equipments on stand-by having pre-selected contractors allow immediate action during the flood season to reduce the amount of damage to the structure.

The blocks of standard size manufactured in factories can be a better source for stockpiling of this type of materials during emergencies. Size and strength of block materials suggested by the designer, may be stock piled at vulnerable locations. Quality of this type of blocks shall be also ensured when manufactured in big plants.

5.3.7 Environmental management

Every stream is different in their characteristics. The bed topography, soil and channel material and riparian vegetation all vary; thus the nature of bank erosion will change with these parameters.

From the environmental point of view the best channelization project is the one which is absolutely necessary and involves the minimum modification of the natural channel. To minimize the environmental degradation associated with channelization, following concepts must be recognized:

- A balanced approach to water resources project is needed. All assets and liabilities must be considered, giving equal weight to environmental values.
- Research must establish the physical behavior of natural streams precisely, so that the potential impacts of river training works can be better recognized.
- Preference must be given to stream modifications that minimize environmental degradation and maximize the return to water users.

It may be pointed out that any activity that removes vegetation or changes the flow or sediment parameters of a stream will affect the fish and wildlife. Fish and other stream inhabitants generally require a habitat characterized by the following (Keller, 1976):

- A variety of low-flow conditions varying from deep water to shallow water is necessary for feeding, breeding and cover,

- A variety of high flow conditions and shelter areas provide protection from excessive water velocity,

- Due to high and low-water conditions and variable stream velocity, there is a natural sorting of bed-load materials on ripples and point bars; this sorting is helpful in creating a good environment for the bottom-dwelling organisms in stream that fish and other aquatic animals depend on for a food supply,

The Guidelines of the Government of Bangladesh, including the "Guide lines for Environmental Impact Assessment (EIA)", prepared by ISPAN (FPCO, 1992) and subsequently other guide lines suggested by WARPO and GOB may be consulted for the environmental impact assessment. The important environmental components (IECs) in a "without project" situation shall be compared to a "with project" situation. The IECs mainly are:

1. Physical Environment that includes; Climate, hydrometeorology and air; Geology and soils; Physiographic, river Hydrology and Morphology, Surface Water Quality and Ground Water,

2. Biological Environment that includes; Aquatic Ecosystem, Terrestrial Ecosystem and Endangered Species,

3. Human and Economic Development that includes; Land use and settlement, Water supply and sanitation, Agricultural productivity, fish culture and Navigation,

4. Quality of Life value that include; Way of life and equity, Income and poverty, Health and education and Cultural and heritage sites.

Different alternatives with and without the project shall be evaluated and compared using a multi-criteria analysis that summarizes economic, social and environmental consequences.

The basic philosophy of the studies and evaluations is to ensure that adverse impacts as a result of such construction and rehabilitation activities are minimized. In order to achieve this objective all negative impacts have to be identified and mitigated during design and construction.

Anticipated Environmental Impacts and Mitigation Measures for the recommended option shall be worked out to look into the recommended Environmental Mitigation Measures and Environmental Monitoring aspects.

Part – II

Design

CHAPTER SIX

Data Requirements

6.1 Hydrologic and Hydraulic Parameters

The parameters related to hydrologic and hydraulic conditions are discharge, water levels, current or velocity and waves.

The lifetime of a protection structure has often been related to the recurrence period of the design flood which can be taken as 30 to 50 years for a bank protection structure. Important protections works can be designed for 100 years return period.

For the design of bank protection structures the maximum and the minimum water levels are important. Therefore, SLW (Standard Low Water), SHW (Standard High Water) and DHW (Design High Water) are defined for the location of the protection site. The definition of SLW and SHW in Bangladesh is the 5% non-exceedence level and 5% exceedence level, respectively, for non-tidal rivers. The DHW can be defined as the maximum water level with a return period of 25 to 100 years for the bank protection structure.

Generally, the selection of a return period of one parameter in one location is not sufficient to characterize a hydrologic event as a typical design event. For example, the maximum discharge of 1988 flood had a return period of 100 years, but the maximum water level in a protection site in the Jamuna was not so extreme and had a return period of 60 years.

An extensive network of hydrological stations in the major river catchments in Bangladesh has been established by the Bangladesh Water Development Board (BWDB), which is the main source of hydrological data in Bangladesh. With a frequency analysis of the available data, the design water levels and the design discharges are determined for the site of interest. Once the design water level is found out, wave run-up and wind set-up have to be added.

For calculating the water levels of a river between the gauging stations and for calculating the levels for defined return periods statistical packages are used in some projects.

6.2 Morphological Parameters

General parameters related to morphology of rivers are slope, width, mean depth, particle size, sediment transport, bankfull discharge, dominant discharge etc. Other important parameters are:

(i) Planform characteristics
(ii) Dynamics of the river system : number of channels, chars etc.
(iii) Shifting and widening of the river
(iv) Bend characteristics and cutoff
(v) Channel cross-section and scour depth
(vi) Location of other bank protection or river training structures in the vicinity of the site including the Pucca ghat

The most severe loads on the structure are not related to the most extreme flood events. 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. Considerations for extra structure induced scour and extreme oblique flow attack due to planform development are also required.

General surveys prior to the planning phase serve the best approach to derive information on the bankfull channel width, the expected water depths and the cross-sectional shape. Bangladesh Water Development Board normally collects data of the major rivers at selected cross-sections during the lean season. Cross-sections should be monitored at least at bankfull flow conditions to represent morphological design conditions. Morphological changes to water levels higher than FPL (Flood Plain Level) are negligibly small. Even if local erosion of the river bed during flood events produces higher water depths below high flood level, in other locations sedimentation will occur so that the maximum depth in the cross-sectional profile does not change significantly. The bankfull channel width and additionally the bend curvature can also be obtained from satellite images and planform analysis.
It may not be possible to survey cross-sectional profiles during hydraulic conditions that represent the morphological design conditions. A field observation during flood period may give an idea of the morphological design conditions.

6.3 Schematization of Cross-section

Straight River Sections

The size of the channel can be estimated using a characteristic width and depth according to empirical regime formula (RSP Special Report No.6, FAP 24, Delft Hydraulics and DHI, 1996 a). The design channel is idealized by a rectangular cross-section as indicated in Fig. 6.1.

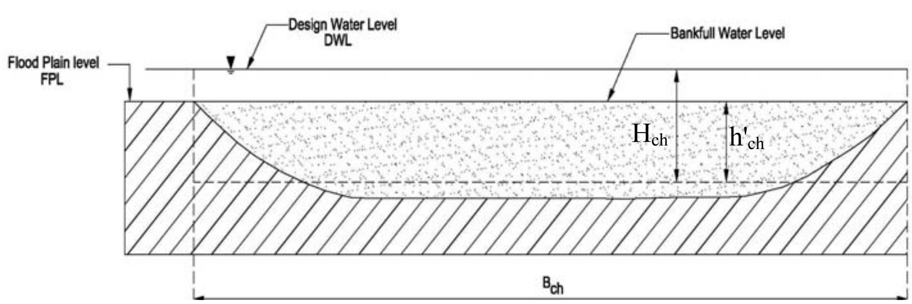


Fig. 6.1 Sketch of a normalized cross-section in a straight river section

The cross-sectional area of the channel is defined as:

$$A_{ch} = B_{ch} \cdot H_{ch} \tag{6.1}$$

with

A_{ch} (m^2)	area of the channel cross-section
B_{ch} (m)	channel width between bank lines of bankfull discharge at FPL
H_{ch} (m)	average water depth within the channel at DWL

h'_{ch} is the average water depth within the channel related to FPL (Flood Plain Level). The average channel width B_{ch} at FPL and the average bankful water depth h'_{ch} are calculated from bankful channel discharge $Q_{b,ch}$ by regime relations:

$$h'_{ch} = C_1 Q_{b,ch}^{C_2} \tag{6.2}$$

$$B_{ch} = C_3 Q_{b,ch}^{C_4} \tag{6.3}$$

The C_1, C_2, C_3 and C_4 are empirical coefficients, the values of which are listed in Table 6.1 below:

Table: 6.1 Values of C1, C_2, C_3 and C_4

Coefficient	Jamuna	Ganges	Padma
C_1	0.40	0.28	0.28
C_2	0.26	0.29	0.30
C_3	8.97	9.97	4.76
C_4	0.57	0.555	0.62

The average water depth H_{ch} below DWL (Design Water Level) is calculated from the bankful water depth h'_{ch} and the elevation of DWL and FPL as follows:

$$H_{ch} = h'_{ch} + (DWL - FPL) \tag{6.4}$$

Finally, the elevation of the riverbed can be calculated by subtracting the water depth H_{ch} from the elevation of DWL.

River Sections at Bends

In case of river bends, the defined bed width B_{ch} of the channel does not cover the total width of the river (Fig. 6.2). Instead, the inner bankline is defined by chars, which are dry at low water period but are inundated at bankfull water level. Consequently areas with shallow water are neglected.

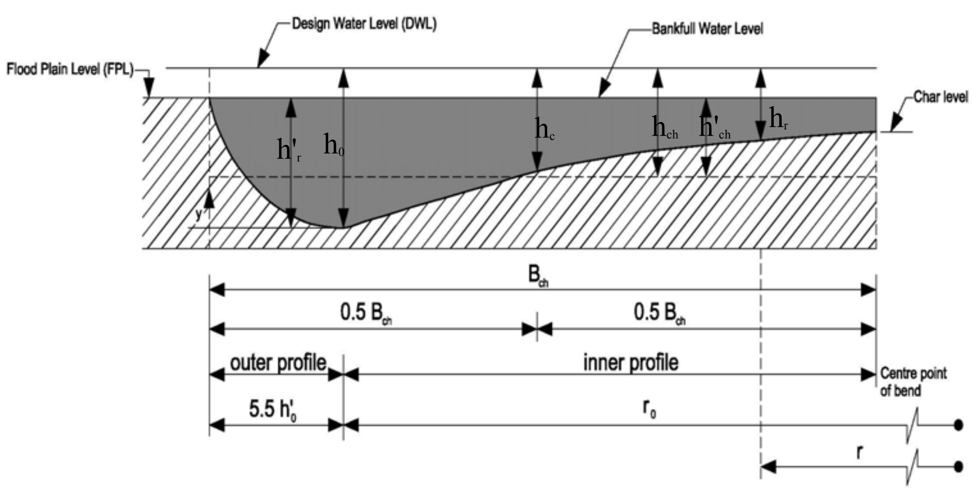


Fig. 6.2 Typical cross-section of a bend.

Assuming that the water depth in the upstream straight river section equals the cross-sectional average water depth h_{ch}, the total water depth at thalweg for $r_c > 1.5B_{ch}$ is given by empirical equation of Thorne 1988 (see Eq. 7.7).

6.4 Survey

The framework of each river survey should be set up considering factors such as:

- extent of the area involved,
- Types of data to be collected,
- density of the measurement points,
- frequency of the measurements,
- period of time allocated,
- timing of the measurements in relation to the regime,
- accuracy required, and
- method of data processing.

Cross-sectional Survey

The Morphology and Research Circle of BWDB surveys standard cross-sections in the lean season extending from November to May. Their survey of a cross-section comprises two parts as: land part survey and wet part survey. The aim of these surveys is to characterize the morphologica1 changes in a river over time.

In the land part of the survey BWDB uses measuring tape, pegs, theodolite, level, level staff, iron flat etc. for measuring distance and corresponding ground elevation. In the river part of the survey the water depth is measured by an echo-sounder mounted on a survey vessel. Sometimes a sounding lead is utilized instead of an echo-sounder depending on the river and the water depth. For fixing the position in a river, generally a theodolite or sextant and land markers are used. In the water part, to identify the survey line, two flags mounted on two bamboo poles of different height are used.

BWDB standard cross-sections are generally defined by three to five monuments or pillars. Pillars are 100mm by 100mm in size, and also made of grounded concrete with an elevation of 0.10 to 0.15 m above the ground level. The alignment of a cross-section is fixed by a bearing with respect to magnetic North. Once the location and alignment of a cross-section are established they remain the same, unless major planform changes make it difficult to survey the section.

Except BWDB regular surveys, local cross-sectional measurements are carried out under different projects, mainly by echo-sounder mounted on a survey vessel aided by a DGPS system.

Bathymetric Survey

To find out changes of bed configuration in a large area, bathymetric surveys are carried out. During a typical bathymetric survey, the survey boat is equipped with a GPS, remote receiver, telemetry receiver, echo-sounder and a navigation computer. The GPS remote receiver tracked all GPS satellites with a view to calculating the present position, which is corrected by the differential correction received from the GPS reference station. The transducer transmitted an acoustic pulse generated by the echo-sounder, which is reflected at the riverbed and received by the echo-sounder to derive the actual water depth by the travel time of the pulse. Both data, the DGPS position and the water depth, are interfaced to a navigation computer. The data are generally recorded by hydrographic software.

Track lines covering the specified area are defined before actual survey starts. To follow straight lines by the survey boat, the selected track line is displayed on an external monitor. The display provided the helmsman with all navigation data to maintain the track.

River Monitoring and Bankline Survey

River monitoring provides the data for planning of active interventions. Large-scale bathymetric surveys, undertaken at the end of the monsoon season, can be used to verify the morphological prediction of river changes and provide baseline data for the work during the dry season. Important additional information collected are (i) maximum river depth, (ii) flow velocities, and (iii) angles of attack on protective works (through float tracking and ADCP measurements).

Data collection allow assessing (i) changes of the bankline, (ii) river conditions during construction phases, (iii) riverbank slopes, (iv) the development of falling aprons used as toe protection or launching heaps used as immediate protection in case of emergencies. All post-construction data provide the background for the planning of adaptive works. In addition these surveys allow assessing the risk for surpassing navigation limits or in case of dredging the risk of damages to the established work

Bankline surveys are based on a dense survey grid. River monitoring targets at large-scale changes, for example the locations of channel or the movement of deeper scour holes and as such can depend on a few survey lines per kilometer. Bankline surveys, on the other hand, depend on a dense grid of information to observe also the beginning of changes resulting from smaller damages, that undetected, could lead to the partial collapse and major repair requirements. Commonly 10 to 25 m line spacing is applied, in special cases 5 m line spacing is necessary.

The adaptive approach of bank protection depends on monitoring throughout the year. While monitoring is not that important in many places during the dry season, flood season monitoring is very important. Proper instrumentation and survey fleets or vessels are required for regular surveys along the protected banks and for large-scale river surveys. The information is also useful to support prediction methods. Survey vessels with dual frequency echo-sounder, RTK-Positioning system, and ADCP (Acoustic Doppler Current Profiler) are currently used in some projects. Measurements with RTK have an overall accuracy of a few centimeters and provide precise data for monitoring changes along protected bank lines.

CHAPTER SEVEN

Design Considerations for Bank Protection Works

7.1 General Approach

The philosophy of bank protection structure should be to design and construct structures at such a level that certain degree of damages is allowed during occurrence of the extreme events. On the other hand, design shall be such to exclude total failure of the structure to avoid wastage of money. Any damage occurring during extreme events may be taken care of through monitoring and repair. This will allow the designer to arrive at an optimum design, which would require a thorough knowledge of the loads, the physical processes and the interaction of different factors.

Bank protection works are essentially important parts of river training works. Out of all types of training structures, bank revetments and groynes are highlighted in the guide lines as use of these two types of training works are frequent in Bangladesh. The purpose of bank revetments and groynes is to prevent bank erosion that means they have to be designed to resist the current and wave and to protect the river bed and river bank against changes. Their principal function is to provide a stable river bank. To fulfill this role they must meet certain basic requirements out of which stability is of utmost importance. Construction methods, construction materials and dimensions of the selected structures depend mainly on the hydraulic loads acting on the structures. They must be capable of withstanding the imposed loads and must have the necessary strength characteristics to resist displacement.

For design of groynes and bank revetment structures relevant mechanical properties of soil and hydraulic loads must be known. Data on waves, discharges, currents and water levels need to be converted to loads acting on the bank protection structures.

7.2 Design Criteria

The design criteria in this section are the hydraulic and hydrologic design parameters for bank protection. These design parameters are the design discharge, the maximum, minimum and standard low water levels, flow velocities, wind speed and wind waves, design scour depths and morphologic changes.

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 in major rivers.

The discharge of the design flood is defined in order to estimate the Design Water Level (DWL) with a selected return period. 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 is negligible.

Traditionally, design conditions, as described above, are related to extreme discharges with a certain prescribed return period, assuming that the most severe loads on hydraulic structures occur at the highest discharges. However, the most severe conditions for bank erosions may occur at lower discharges around below bankfull stage, for example: (i) at certain unfavourable near-bank channel configurations, such as deep channels, channel confluences and sharp bends; and (ii) following rapid scour hole development or a rapid fall of the water level.

Return periods for the occurrence of the unfavourable near-bank channel configurations would be a better basis for design than return periods for extreme discharges. The problem is that the bank protection structures themselves make these configurations more unfavourable.

7.2.1 Water Levels

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

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

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

Low Water Level: Lowest During Dry Season (December-March)
Average high in the dry water months (December-March)

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

In Bangladesh lowest water level is attained in different months in different river systems. Ganges at Rajshahi attains lowest water level by the end of March and ranges between 7.90 m to 10.30 m, PWD. Brahmaputra-Jamuna at Bahadurabad attains lowest water level by end of February or early March and ranges between 12.89 m to 13.75 m, PWD.

Standard procedures use low water level (LWL) as important reference for pitching of slope protection of bank from LWL to HFL (High Flood Level) or FPL (Flood Plain Level), and also for computing D_s (depth of scour from DLWL) for computing length of falling apron which is normally taken as 1.5 D_s. LWL also forms the basis for selecting the construction window for bank protection works.

A construction window has been defined as the number of days during which the average water level is below a certain level with certain probability. From the flow-duration curve the level not surpassed during periods of 60, 90 and 120 days can be extracted for the selected frequencies. Normally, the period from December to March is considered to be the construction window for all bank protection works because this period is the most propitious for such construction in Bangladesh and water level generally maintains a level which is below certain water level as mentioned above.

Following points may be considered in fixing the LWL:

- High value of adopted LWL will decrease the length of slope protection but will increase the value of D_s that will increase the length of falling apron,
- Adaptation of too low value of LWL may create a situation that the level may not be achieved in the year of construction.
- The idea for time available for construction (construction window) should therefore be such that some flexible planning may be done well before the construction period.

Selection of DLWL:

(I) Non Tidal Area

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

(II) Tidal Area

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

7.2.2 Flow velocity

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

- Design flow velocities are obtained either from field measurement or in a physical model investigation. These design flow velocities depend on the approach flow and on the alignment of bank protection structure,
- Average flow velocity in a cross-section of a channel is estimated with a suitable equation.

The flow velocity obtained from the method stated above shall be verified with the observed measurements available for that stretch of the river. For the design of drag force on the piles and bed protection around the piles of permeable groynes a designed flow velocity is defined as the upstream flow velocity which is not influenced by the permeable groynes. In general the depth averaged flow velocity is used.

The flow attack on a revetment depends not only on the discharge and the water level but also on the alignment of approach channel. The design flow velocities are the maximum flow velocities measured on the physical model with the extreme alignments of the approach channel.

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

$$\bar{u} = 0.095\ Q^{0.26} \tag{7.1}$$

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

$$\bar{u} = 0.36\ Q^{0.15} \tag{7.2}$$

Where $\bar{u}$ is the average velocity and Q is the bankfull discharge.

For a bankfull discharge of 44,000 m^3/sec, this results in an average velocity of about 1.5 m/sec for Jamuna and 1.8 m/sec for the Ganges. Locally the velocities may be much higher due to dimensional effects. Maximum velocities measured were (FAP-24):

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

In absence of any field or experimental data, it is reasonable to assume the design velocity as twice the upstream mean velocity to determine the size of revetment at the nose of groynes (BWDB, 1993). However, in the past BWDB used the design velocity ranges between 3 and 4 m/s for cover layer design in different river bank protection works (BWDB, 2002). Effectiveness of some of these structures was found to be satisfactory as far as the hydraulic load due to current attack is concerned.

7.2.3 Design Discharge

The design discharge may be taken from available analyzed records or may be computed from available data. In case of ungauged basin the same may be computed by simple synthetic unit hydrograph method or complex mathematical model.

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

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 braided channel can be described by:

$$Q_{ch} = \frac{c_1}{c_b} \times 2 \times Q_b \qquad (7.3)$$

where,

Q_b	(m^3/s)	bankfull river discharge
c_1	(-)	safety factor for extreme channel discharge, $c_1=1.5$
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 for Jamuna, Ganges and Padma can be taken from the following table:

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

The bankfull discharge $Q_{b.ch}$ for a channel is calculated by:

$$Q_{b.ch} = \frac{Q_b}{c_b} \qquad (7.4)$$

Where

$Q_{b.ch}$ (m^3/s)	bankful channel discharge
Q_b (m^3/s)	bankful river discharge
c_b (-)	braiding index

The main parameters on discharge and water levels used in design of Jamuna Bridge are summarized as:

Return period	Q [m^3/s]	Water level [m,PWD]
Average annual flood	65,000	13.7
1:10 year flood	76,000	14.3
1:50 year flood	87,000	14.4
1:100 year flood	91,000	15.1

7.2.4 Waves

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

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

The principal factor affecting the design of slope protection is wave action. The mechanics of wave generation are extremely complex, and the forces causing erosion during wave attack on an earth slope are both varied and complex. The described ranges of riprap design assume that the wave height is a direct measure of the erosiveness of the wave attack (Fig. 7.1).

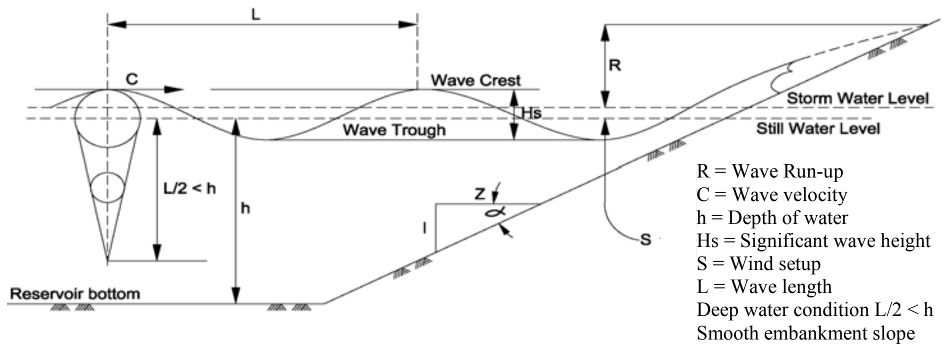


Fig. 7.1 Illustration of wave definitions

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

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

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

7.2.4.1 Wave Analysis

(a) General: When wind blows over open water, the water surface starts to move and form waves. Waves are the result of shear stress between moving air (wind) and water surface. The energy of the moving air is transferred to water particles. These water particles start to move in the same direction the wind blows.

Waves are defined by wave height H, wave period T, wavelengths L, and direction (see Fig. 7.2). Waves are often generated far from the place where they are observed. However, related wind speed and duration can be derived from the observed wave height, wave period and wave direction. Uncertainties in the relationship between wind and waves result from the fact that waves can be transferred and reflected during travelling. If the waves are travelling through areas with shallow water they are in contact with the bottom and start changing direction and height called shoaling and refraction. The calculation remains simple as long as the waves travel in the deep-water zone ($d > 2 \cdot L$, where d is the water depth and L the wave length). Due to refraction and shoaling the wave height is decreasing and the wavelength is changing. The wave period remains the same.

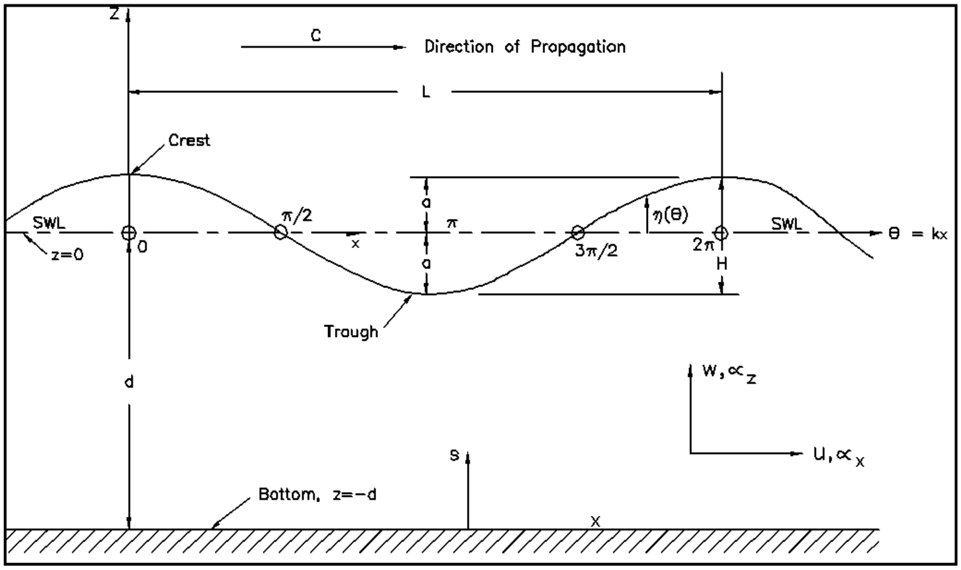


Fig. 7.2 Definition of terms – elementary, sinusoidal, progressive wave (CEM 2003)

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

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

Figures 7.3 and 7.4 show diagrams for estimating wave height and period under fetch-limited condition. It shows, for example, that the generated wave for a 5 km wide water surface (fetch) and 15 m/s wind speed is 60 cm high.

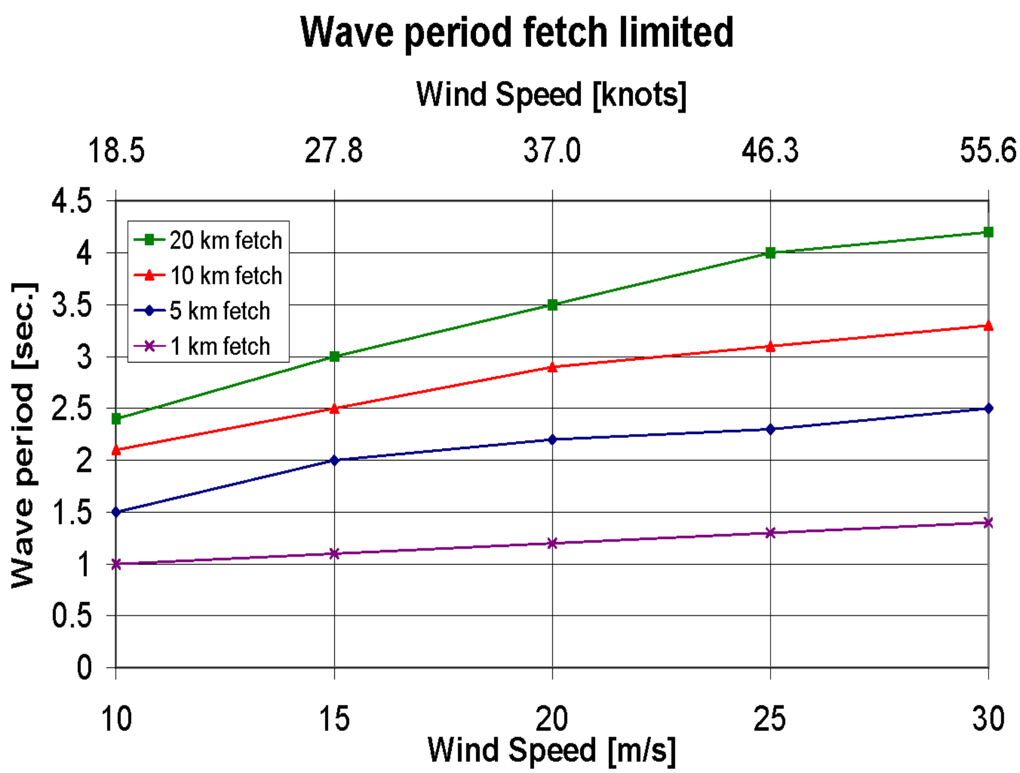


Fig. 7.3 Wave period for fetch limited wave climate.

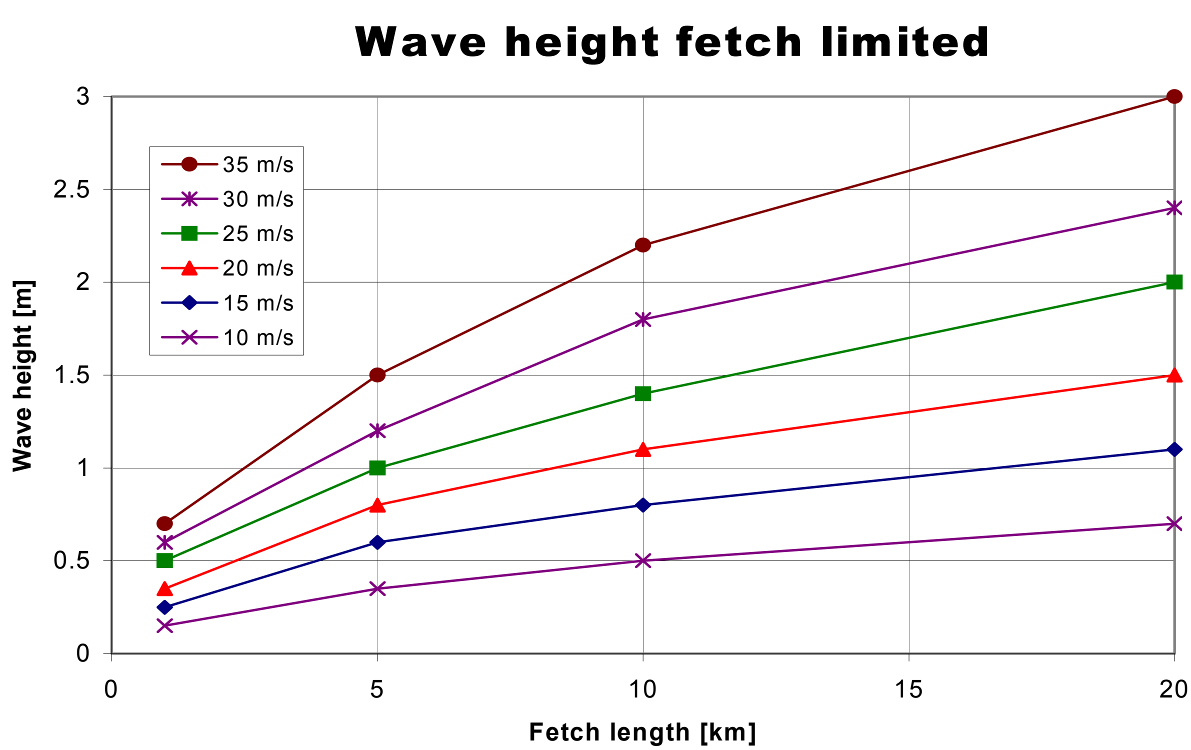


Fig. 7.4 Wave heights for fetch limited wave climate.

7.2.4.2 Data for calculating wave heights

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

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

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

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

Significant Wave Height (H_s): The significant wave height is the average height of the highest one-third of the waves for a specified period of time.

7.2.4.3 Waves in Rivers

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

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

7.2.4.4 Wind and wave environment in Bangladesh

Bangladesh is located in an area with a high risk of thunderstorms. These thunderstorms can generate fast winds. The generated wind can only create high waves, if the storm duration is long and direction is such that a long fetch sufficient to generate high wave is maintained.

Halcrow investigated wind data in 1994.The maximum wind speed equal to or exceeding 30 knots recorded was as follows:

Table 7.1 Wind speed and direction reported by Halcrow (1994):

Location	**Wind speed [knots]**	**Wind speed [m/s]**	**Direction [°]**
Faridpur	35	18.0	340
Faridpur	30	15.4	150
Sirajganj	35	18.0	150
Bogra	30	15.4	200
Bogra	30	15.4	360
Bogra	30	15.4	130
Mymensingh	30	15.4	50

Halcrow estimated a maximum wave height of 1 m with a period of 3.0 sec. Unfortunately Halcrow gives no information about the duration of the wind speed.

To generate wave heights of 1 m as reported by Halcrow a wind of 30 knots (15m/s) blowing over 20 km fetch length for about 3 hours is necessary. A wind of 15 knots (8 m/s) is necessary for about 9 hours. The wave period is then 3.5 sec.

In Jamuna Bridge design two sources of major wind speeds were taken into account:

- Violent thunderstorms squalls (called “Nor-Wester”),
- Tropical cyclones originating from Bay of Bengal.

Analysis of the wind data revealed that thunderstorms dominate; the gust wind speed can be computed by;

$$V_{thunderstorms} = 30 + 4.5*\ln(T) \qquad (7.5)$$

Where, $V_{thunderstorm}$ [m/s] wind speed
T [years] return period

A wind duration of 15 minutes and a water depth of 10 m over a fetch length of 4000 m in front of the structure result in the following wave height:

Return period (year)	Wind speed (m/s)	Significant wave height (m)
2.3 (annual average)	15.2	0.68
10	20.4	0.93
100	25.6	1.19

The maximum wave heights along the major rivers are generated by squalls during nor-westers. A design wind speed is defined as the maximum wind speed which is averaged over a period of 15 minutes.

Maximum wind speed of 12 m/sec with a frequency of once every 2 to 4 years and 18 m/sec with a frequency of once every 5 years are to be expected along the Jamuna River (BRTS, 1991).

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.

Maximum wind speeds are mainly generated 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 bank lines 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 regarding channel width can be obtained from satellite images.

7.2.5 Scour

Scouring is a natural phenomenon caused by the flow and turbulence of water in the river. Scour relates to deeper parts of the river caused by specific flow patterns, most of the time imposed by the planform of the river. The following types of scour have been identified for the main rivers of Bangladesh:

- Confluence scour,
- Bend scour,
- Protrusion scour
- Local scour

Two approaches have been outlined below for the estimation of design scour depth. These are:

(A) Scour calculation based on local geometry of the river and
(B) Lacey's regime approach.

(A) Scour calculation based on local geometry of the river:

The first method is based on the increased understanding of river processes and local scour phenomena as practiced in FAP 21/22. This method attempts to distinguish between different types of scour, and for each type of scour a quantitative estimate is made. In the second step the different values are added to arrive at the combined scour depth that will actually occur. Some of the scour types under this method are discussed below.

(i) Confluence Scour

Confluence scour occurs downstream of the confluence of the two channels. The confluence scour is characterized as an elongated reach of relatively high depth in the middle of the combined downstream channel. The probable reason for occurrence of scour hole at the confluence is back-to-back bend scour as caused by helical flow generated by induced curvature of the stream lines or increased turbulence due to the bouncing of water of two confluencing channels (Mosley, 1976; Ashmore and Parker, 1983). Klaassen and Vermeer (1988) suggested the following empirical relation for estimating the confluence scour:

$$y_{s,\,confluence} = h_1' \,(0.292 + 0.037\ \varphi) \tag{7.6}$$

where, φ (°) denotes angle of incidence of two confluencing channels

h_1' denotes average water depth of two channels below SWL

(ii) Bend Scour

Bend scour develops at the outer part of a river bend and the scour hole is formed by the secondary flow associated with the curved flow. The bend scour depth $y_{s,bend,}$ which represents the deepening of the riverbed along the thalweg, is a decisive parameter.

Thorne (1988) found an empirical formula for h_o, r_c and B_{ch}. Assuming that water depth in the upstream straight river section equals the cross-sectional averaged water depth h_{ch}, the formula reads:

$$\frac{h_o}{h_{ch}} = 2.07 - 0.19.\log\left(\frac{r_c}{B_{ch}} - 1.5\right) \tag{7.7}$$

in which

B_{ch} =channel width in m
h_o = total water depth at thalweg below DLWL (Design Low Water Level) in m
r_c = radius of bend centerline; ($r_c > 1.5B_{ch}$) in m

with $r_c = 2.5B_{ch}$, the equation (7.7)comes down to:
$h_o = 2.07h_{ch}$

The scour depth $y_{s,bend}$ 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 of the scour hole. So the bend scour depth $y_{s,bend}$ can be written as:

$$y_{s,bend} = h_o - h_{ch} \tag{7.8}$$

(iii) 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. The following formula (Richardson et. al,1975) is recommended to be used:

$$\frac{y_{s,\,protrusion}}{h_1} = 4Fr_1^{\;0.33} \tag{7.9}$$

in which Fr_1 is the Froude Number = $u_1/(gh_1)^{0.5}$, u_1 is the upstream depth averaged velocity, h_1 is the upstream depth of flow and g is gravitational acceleration.

The actual protrusion scour depends on local geometry and flow patterns. However, protrusion scour is not the decisive phenomenon for the design of a protective structure. The downstream local scour depth induced by the structure is of much higher importance.

(iv) Local scour

Local scour is caused by the increased turbulence and the vortices in the decelerating flow downstream of a structure. The depth of the structure induced local scour in a concave bend of sandy cohesionless riverbed (applicable for the major rivers of Bangladesh) can be determined by the formula (Ahmad, 1953):

$$h_1 + y_{s,local} = K\left(h_1 \cdot u_1 \cdot \frac{B_{ch}}{B_{ch} - b}\right)^{\frac{2}{3}} \tag{7.10}$$

in which

K ($m^{-1/3}s^{2/3}$)	empirical coefficient
B_{ch} (m)	channel width upstream of the structure
b (m)	protrusion length
u_1 (m/sec)	upstream depth averaged flow velocity
h_1 (m)	water depth upstream from scour hole
$y_{s,local}$ (m)	local scour depth.

The empirical coefficient K has to be chosen individually according to the type of structure; values of K for groynes and revetments shall be taken from section 7.2.5 (iv) (a) and (b).

Similar to Ahmed's formula later Neil (1973, 1980, after, Przedwojski et. al 1995) introduced sediment size as variable in the following form:

$$h_1 + y_{s,local} = K'.(d_{50})^{-0.102}\left(h_1 u_1 \cdot \frac{B_{ch}}{B_{ch} - b}\right)^{\frac{2}{3}} \tag{7.11}$$

Here d_{50} is in millimeter. Neil formula yields higher value of coefficient K' (2.85 to 3.75) than K in Ahmad's formula, when compared with sediment size of Jamuna river. It is to be mentioned here that this formula can be applicable to other rivers using the local bed sediment size.

The ratio $B_{ch}/(B_{ch}-b)$ considers the constriction due to 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. It has also to consider the permeability of the respective groyne section. If the length can not be established, Figure 7.6 can be used to estimate the factor $B_{ch}/(B_{ch}-b)$ in respect of water depth h_{ch} and 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 geometry of the structure if the depth averaged flow velocity u is larger than $2u_{cr}$, where u_{cr} is critical velocity. 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.

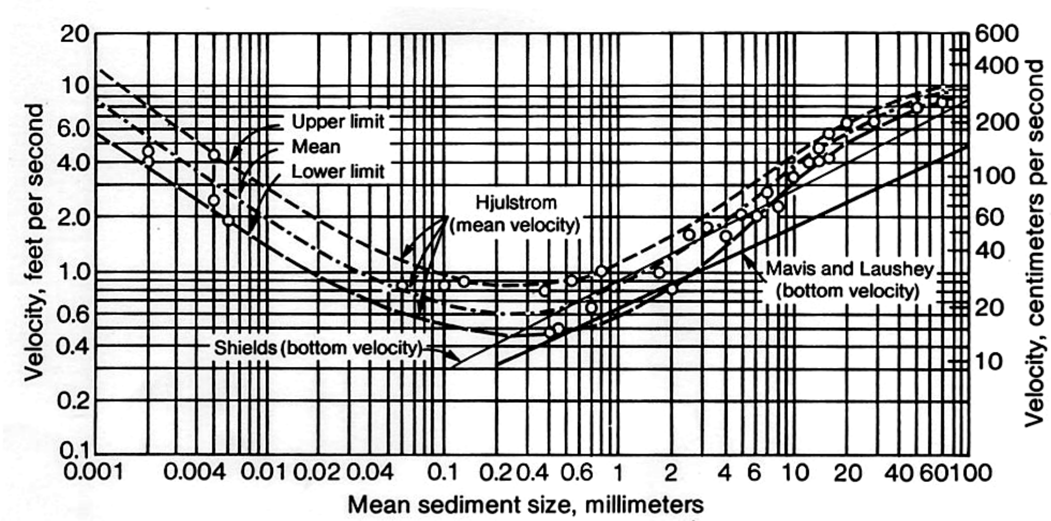


Fig. 7.5 Critical Velocity Range Versus Sediment size (after Chang, 1992)

Therefore, the following limits of critical velocity (u_{cr}), also shown in Figure 7.5, may be considered:

$$u_{cr} \leq u_1 \cdot \frac{B_{ch}}{B_{ch} - b} \leq 2u_{cr} \tag{7.12}$$

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 bend it is represented by water depth h_o in the thalweg. This strongly depends on the width of the bend scour related to the size of the local scour hole.

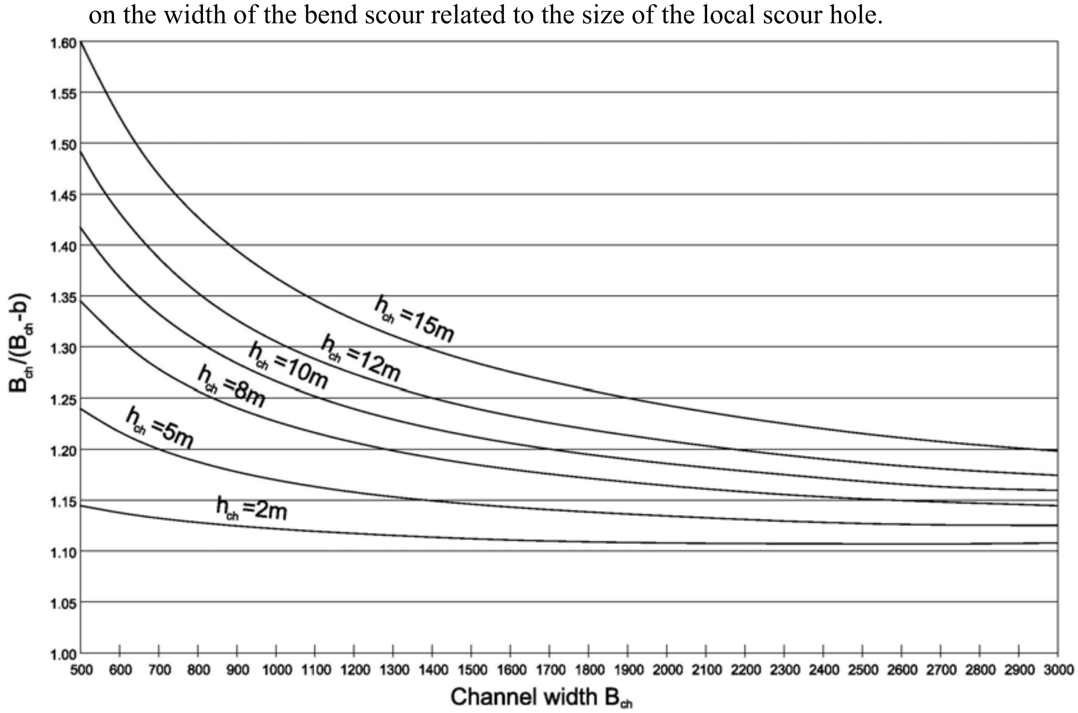


Fig.7.6 Estimated constriction factor for local scour calculation

(a) Local Scour at Groynes

For a permeable groyne, bend scour can interact with local scour at the tip. In theory, also an interaction between confluence scour and local scour, or confluence scour, bend scour and local scour is possible, and might result in deep scour. Therefore, the design procedure uses the total design scour depth.

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.

$$K_{total} = K_{structure} \cdot K_{bed\ protection} \cdot K_{floating\ debris}$$

Values for the single factors can be taken from Table 7.2:

Table 7.2 Empirical coefficient K for groynes

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
	$K_{bed\ protection}$ (-)
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

(b) Local Scour at Revetment

The local scour in front of and downstream of a revetment can be treated analogous to the local scour at the abrupt transition at the tip of an impermeable structure (Table.7.2) in combination with bed protection. It is therefore, recommended to use Ahmad's formula with K = 2.6. Utilizing sediment size of revetment location, Neil's formula may also be used for comparative assessment. Floating debris does not have to be considered.

(v) Extra Structure Induced Scour

Some of the 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 local bank stabilization. Channels become narrower, bends may become sharper and confluence 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 scour enhancement due to these structure-induced morphological changes is still largely unknown. The extra scour occurs typically over the whole length of the protection structure or over the whole length of the 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 purpose of design, it is therefore, recommended to increase the total scour by 20% to account for the extra scour.

(vi) Total Scour

Total scour results from the combination of the different types of scour. 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 can be added to each other (summation principle). However, the summation principle applied to confluence scour and local scour might result easily in an overestimation of the total scour depth.

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 1.2.

$$Y_{s,ch} = 1.2 * \left\langle Y_{s,bend} + \max\left(Y_{s,local} ; Y_{s,confluence}\right)\right\rangle \tag{7.13}$$

$$Y_{s,o} = 0.2 * Y_{s,bend} + 1.2 \cdot \max\left[Y_{s,local} ; Y_{s,confluence}\right] \tag{7.14}$$

With $y_{s,bend} = h_o - h_{ch}$

(B) ***Lacey's Regime Approach:***

Lacey's regime formula is widely used in this sub-continent to find out scour depth in unconstricted alluvial rivers. This empirical regime formula is:

$$R = 0.47\,(Q/f)^{1/3} \tag{7.15}$$

With $D_s = XR - h$

Where,	D_s	(m)	Scour depth at design discharge
	Q	(m^3/s)	Design discharge
	h	(m)	Depth of flow, may be calculated as (HFL-LWL)
	f	(-)	Lacey's silt factor = 1.76 $(d_{50})^{1/2}$
	d_{50}	(mm)	Median diameter of sediment particle.
	X	(-)	Multiplying factor for design scour depth(Table 7.3)

The water surface level corresponding to Q must be known in order to determine scour level from the above equation. The above equation gives only an estimated mean depth across the channel section. To estimate the maximum natural scoured depth, a multiplying factor must be used as shown in Table 7.3.

Table 7.3 Multiplying Factors for Maximum Scour Depth by Lacey's approach

Nature of location	Factor (X)
Straight reach of channel	1.25
Moderate bend	1.50
Severe bend	1.75
Right angle abrupt turn	2.00
Noses of piers	2.00
Alongside cliffs and walls	2.25
Noses of guide banks	2.75

It is to be noted here that the designer should consider above mentioned two approaches (A) and (B) to estimate the design scour depth. Observed data, if available, can also be used to compare the calculated value to determine reasonable design scour depth.

Observed scour depth, for example, at the upstream side of Sirajganj hard point was found to be -33 m PWD which caused severe damage during the flood of 1998. In 2005, this scour depth attained at -40 m PWD. Also extremely deep scour (-) 40.00 m PWD occurred at Sailabari groyne in the river Jamuna. In designing Jamuna Bridge guide bundh maximum scour depth of magnitude of 40 to 45m was taken. At Tajumuddin, Bhola in the river Meghna, maximum scour depth at the end of the revetment was considered as -32.50 m, PWD.
.

7.2.6 Remarks on Design Scour Depth

Assumed scour development is of major importance for the design of a riverbank protection structure. The tools for the estimation of scour in design are empirical formulas, physical modeling and mathematical modeling.

Empirical formulas developed by different authors to estimate scour depth are mostly for one type of scour though different types of scour have to be considered. The results of different formulas have to be considered to yield logical results. The major problem with these types of formulas is the selection of design flow velocity. As the range of flow velocity distribution in meandering/braided channel of a river is very wide, so the representation of the flow velocity by a depth averaged flow velocity in meandering or braided channel in different formulas may not produce the actual field scenario. Therefore, the design values for the scour depth should be selected carefully using design experience and judgment.

CHAPTER EIGHT

Design of Revetments

8.1 General Aspects

A revetment usually consists of three components: cover layer, filter layer and toe protection. The cover layer must be able to resist hydraulic impacts (current and wave). The intermediate layers between cover and core material are required for drainage and filtering to allow for a stable foundation of the overall system. The toe of a revetment structure requires special attention during design. If overtopping by flow or by waves is to be expected, the stability of the structure crest and inner slope of the revetment also needs appropriate protection.

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

i. The surface of the individual elements of the cover layer should be sufficiently resistant against abrasion by wave and current attack,
ii. The normal weight of the cover layer has to be larger compared to the uplift pressure caused by waves or resulting from excessive pore water pressure,
iii. Revetment should be stable against sliding due to frequent hydraulic loads,
iv. The revetment including filter layer and subsoil must be stable as a whole.

8.2 Layout Consideration

8.2.1 Design

The design of the recommended revetment structure is motivated by construction constraints. In this regard, focus may be put on the (1) construction of all revetment components on the dry flood plain during dry season (December-April). With increasing scour depth in front of the structure, the toe protection elements start to slide down the scour hole, thus protecting the river slope from failure, which is commonly estimated to be in the order of 1V:2H. 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 (average low water level December to April + 0.5 m), or (2) the construction of the revetment structure can start on the existing river section, covering the under water river bank by systematically dropping protective material as per requirement and providing toe protection (falling apron) at the end of the slope to cover the maximum anticipated scour.

Revetments built along the riverbank provide a number of advantages: (i) the land used for construction is minimized, as it only requires to cut back a small strip along the bankline above low water level, (ii) the protection under water follows the alignment of the existing river bank, (iii) the toe protection or falling apron is placed at a deeper level, where more consolidated soils support a more even launching and more stable slopes after launching (Fig. 8.1)

For revetments covering the underwater river bank, systematic dropping of material shall start from the outer edge of the falling apron in the river and proceed towards the water edge near the bank. This type of dropping shall be done in such a way that the under water river bed and bank are covered with a specified uniform layer of protection material. The slope above low water may be covered by placing hard material over a filter layer. The filter should be extended to at least 1.0 m below the average low water level.

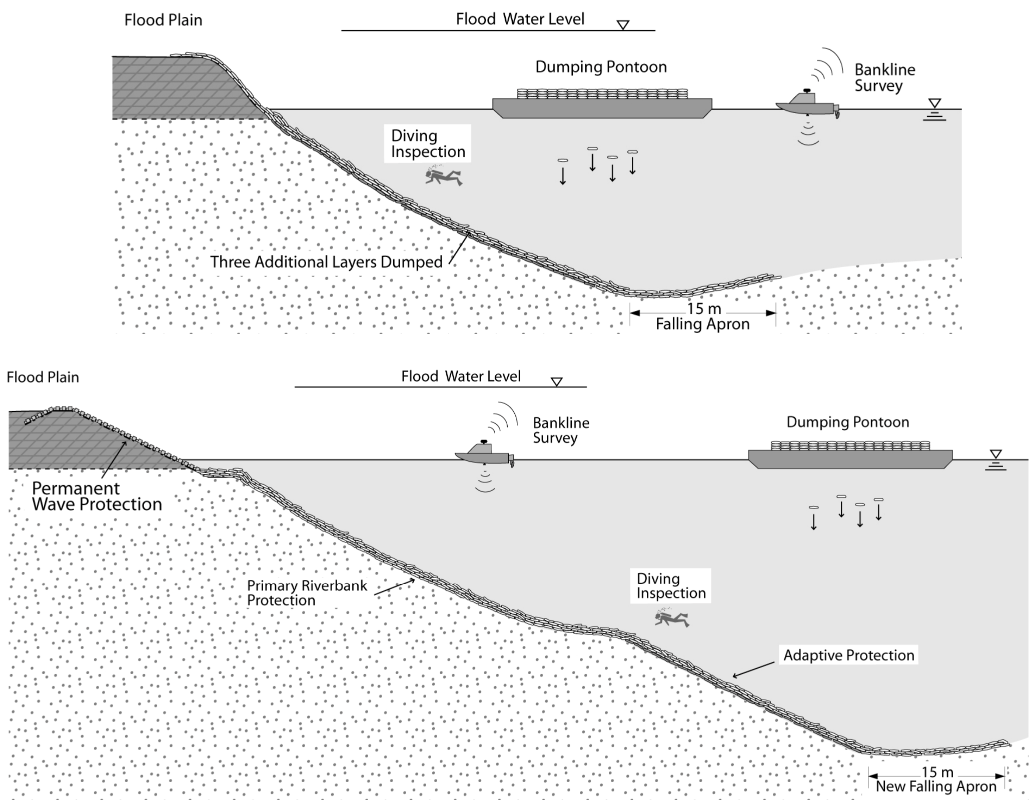


Fig. 8.1 Phased protection of the underwater slope starting from the existing river bank.

8.2.2 Length and Alignment

Length and alignment of a revetment structure must be selected in view of the area to be protected and are best based on morphological prediction. Due to the relatively expensive structure terminations, a certain minimum length should be provided in design. In absence of any design rule for selecting the length, morphological changes and erosion forecast received from the appropriate agencies will be the guidelines for selecting the location and length of the revetment structure. Engineering judgment is the key for final selection of length of protection works.

The angle between the structure front and the course of the main river is also of prime importance. To reduce turbulence due to flow separation, it is recommended to arrange the structure at a small angle (approx. 3 to 5 degrees, Chow 1959) to a parallel line, both, relative to a major branch and to the main course (the downstream end of the structure is pointing towards the channel). The recommended alignment will always involve certain compromises under different and often changing morphological situations.

8.2.3 Cross Sectional Requirement

The slopes to be applied to the revetments of banks and embankments depend on the quality of the subsoil, ground water condition and selected type of revetment. Analysis of slope stability is required in each case. It should be stressed here that the revetment consisting of various components has to be seen as a whole.

8.2.4 Erosion Resistance to Subsoil

For the stability of a revetment the subsoil i.e. the body of the embankment or bank, play an important role. River bed and river bank are subject to current and wave attack causing erosion and scours.

The interaction between structures and the soil and possible failure mechanism are discussed in relevant sections. When exposed to current and wave attack the behaviour of various soil types depend mainly on the amount of cohesive material.

The erosion resistance of river bank and river bed can be estimated on the assumption that the load can be characterized by the maximum occurring shear stress or velocity, while the resistance is given in terms of the critical shear stress or velocity.

The following information gives an indication to the erosion resistance of various subsoils.

(1) Non-cohesive sediment

Two methods of assessing the erosion resistance of non-cohesive soils can be applied:

i. the method of shear stress, and
ii. the method of permissible velocity.

Assuming a steady uniform flow, the basic stability criterion can be expressed as:

$$\tau_{cr} = \Psi_{cr}(\rho_s - \rho_w).g.D > \tau_o \qquad (8.1)$$

where,

τ_{cr}	(N/m^2)	critical shear stress
τ_o	(N/m^2)	shear stress exerted along the bank/bed boundaries
Ψ_{cr}	(-)	dimensionless critical Shields shear stress parameter for specific material
D	(m)	mean grain size
g	(m/s^2)	acceleration due to gravity
ρ_s	(kg/m^2)	density of soil
ρ_w	(kg/m^2)	density of water

The value of Ψ_{cr} depends on particle shape, velocity profile etc. For fine sediments with D< 4 mm, the Shields parameter can be written as:

$$\psi_{cr} = \frac{\tau_c}{(\gamma_s - \gamma_w)D} = \frac{U_*^{\;2}}{\Delta_m g.D} \qquad (8.2)$$

where,

$$U_* = \sqrt{g \cdot R \cdot S} = \frac{u \cdot \sqrt{g}}{C} \qquad (8.3)$$

where,

Δ_m	(-)	$=(\rho_s-\rho_w)/\rho_w$, i.e.,relative density of submerged material
U_*	(m/s)	shear velocity
R	(m)	hydraulic radius or water depth,
S	(-)	slope of energy line,
u	(m/s)	mean velocity
C	($m^{0.5}/s$)	Chezy coefficient = 18log (12h/k_s),
k_s	(m)	roughness parameter
h	(m)	depth of flow.

Since the Shields parameter depends on the state of movement, it can be assumed:

ψ_{cr}	=	0.03 to 0.04	for no movement conditions,
ψ_{cr}	=	0.05	for limited movement of grains
ψ_{cr}	=	0.06	for movement of grains

The roughness value k_s can be estimated as:

k_s	=	(1 to 2)*D	for uniform sediment
k_s	=	(1 to 3)*D_{90}	for graded sediment

where, D (m) mean grain diameter

For practical cases the following formulas can be used to estimate the critical velocity (U_c) causing subsoil erosion:

$$U_c = \sqrt{g \cdot \Delta \cdot D_{50}}\sqrt{\psi_{cr}} \bullet 5.75 * \log \frac{6h}{D_{50}} \quad \text{(for finer sediment)} \tag{8.4}$$

$$U_c = \sqrt{g \cdot \Delta \cdot D_{50}}\sqrt{\psi_{cr}} \bullet 5.75 * \log \frac{2h}{D_{50}} \quad \text{(for coarser sediment)} \tag{8.5}$$

Where h is the depth of flow. As an indication, the critical mean velocity near the bed assuming uniform flow and smooth boundaries with uniform materials in the range of 0.1 mm < D_{50} < 1 mm can be estimated up to 0.2 m/s.

Based on the worst case of sands subsoil the permissible mean velocity for general use can be taken as 0.5 m/s.

(2) Cohesive sediment

Cohesive sediment such as clay usually has high resistance to erosion than non-cohesive sediment. As an indication the following value may be used:

u_{cr} = 0.8 m/s – fairly compacted clay and void ratio 0.5.
= 1.5 m/s - stiff clay with void ratio 0.25
= 2.0 m/s - grassed clay
= 3.0 m/s – grassed clay bank, reinforced

8.2.5 Terminations and Transitions

Special care is required for the design of any transitions, terminations and joints. Erosion damages are often initiated at discontinuities between protected and unprotected reaches of a riverbank. An extension of the revetment beyond the point of active erosion is in general not feasible in highly mobile and meandering or braided rivers. Usually the smoothly curved and sloped terminations at both ends of the structures are recommended, starting from the straight part and pointing away from the river. The minimum radius R_1 for major rivers should not be smaller than 250 m, preferably it should be 400 m, whereas for the downstream termination a radius R_3 of 100 m to 150 m seems appropriate (Fig. 8.2).

Experience shows that damage to bank protection often starts at terminations, transitions and joints. Though the problems are well known, there are no detailed principles or design rules to estimate the desired size of riprap, concrete blocks or cover layer materials at transitions or joints. As a general principle the revetment in transition zones or at joints should be of strength equal to or greater than the adjoining systems. This can be achieved in one of the following ways:

i. Increase the thickness of cover layer at joints and transitions;
ii. Grout riprap or concrete block cover layers with bitumen, and
iii. Use of concrete edge strips or boards to prevent damage progression along the structure.

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

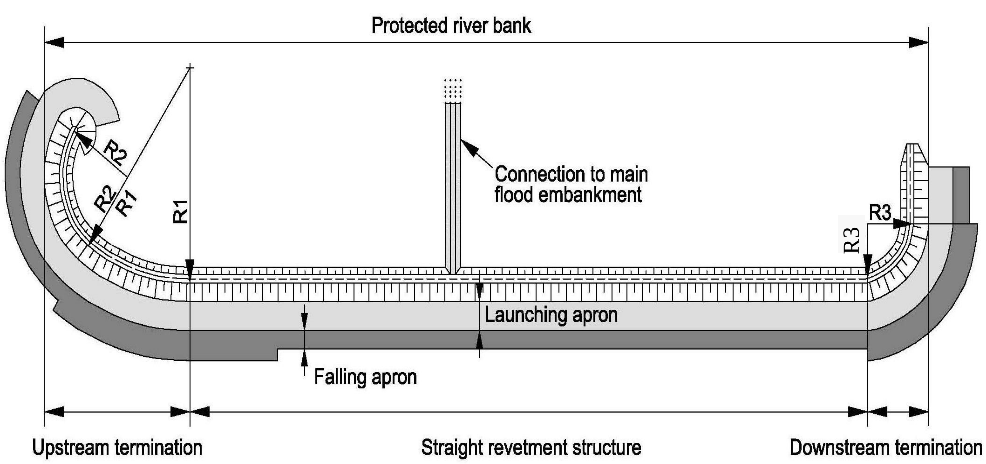


Fig.8.2 Plan view of Typical Revetment Structure.

8.3 Stability of Revetment under Current Attack

8.3.1 Loose Units

Severe flow attack may in practice occur on revetments, such as with flow over a steep slope and flow attack near many kinds of structures (downstream of sills, gates, discharge structures etc). At these structures, the flow is specifically determined by the geometry and the boundary conditions.

Different methods regarding calculations of unit dimensions of revetment cover layers and toe protections (e.g. PIANC, 1987; Pilarczyk, 1990; Escarameia, 1992; FAP 21, 1993) show only marginal deviations within the range of application for the rivers of Bangladesh. Therefore, the widely used Pilarczyk method may be recommended because it includes the turbulence intensity by an empirical coefficient. However, the most common equations relating to the size of the revetment material to the velocity impinging on it is presented for comparison of results under different scenario.

(1) Pilarczyk

With the flow velocity well known, or can be calculated reasonably accurately, Pilarczyk's relation (1990) is applicable. The general formula for the design against current loads is: (Geosynthetics and Geosystems in Hydraulic and Coastal Engineering; Krystian W.Pilarczyk, 2000)

$$D_n \geq \frac{0.035 \cdot \overline{u}^2}{\Delta_m \cdot 2g} \cdot \frac{\phi_{sc} K_\tau K_h}{K_s \cdot \Psi_{cr}} \tag{8.6}$$

Where,

D_n	(m)	Equivalent diameter (cover layer)
Δ_m	(-)	$(\rho_s - \rho_w)/\rho_w$ = relative density of submerged material
ρ_s	(kg/m^3)	density of protection material
ρ_w	(kg/m^3)	density of water
$\bar{u}$	(m/s)	depth averaged mean flow velocity;
g	(m/s^2)	acceleration due to gravity
φ_{sc}	(-)	stability factor for current
K_τ	(-)	turbulence factor
K_h	(-)	depth factor, dependent on the assumed velocity profile and water depth (h) to equivalent roughness height ratio.
K_s	(-)	Slope parameter
Ψ_{cr}	(-)	critical Shield`s parameter,
α	(°)	slope angle of bank or structure
θ	(°)	angle of repose considering the material specific internal friction

The stability of revetment elements also depends on the slope gradient under which the revetment is applied in relations to the angle of internal friction of the revetment. This effect on the stability is taken into account with the slope parameter K_s, which is defined as

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

or, $K_s = \cos\alpha_b$

α_b = slope angle of the river bottom (parallel to the flow)

The following values of angle of repose (Table 8.1) can be assumed as the first approximation:

Table 8.1 Angle of repose θ for various revetment cover layers (Pilarczyk 2000)

Revetment type	θ , angle of repose
Rip-rap	40^0
Sand filled systems	30^0~40^0
Stiff and anchored mortared-filled mattresses and block mats (cabled)	90^0

The angle of repose for the stationary stone suggested by Inglis (1935-37) is shown in Table 8.2.

Table 8.2 Angle of repose suggested by Inglis (1935-37)

	Angle of stationary stones	Angle of rolling stones
Rounded stones laid on similar stones;	30° = 1V:1.3H	31° = 1V:1.7H
Angular stones laid on similar stones;	40° = 1V:1.2H	34° = 1V:1.5H
Rounded stones laid on Ganges sand	31° = 1V:1.7H	25° = 1V :2.1H

The equivalent diameter D_n can be estimated as follows:

- For rock: $D_n = (M_{50}/\rho_s)^{0.33}$ or $D_n = 0.84\ D_{50}$ and $\Delta_m = \Delta = (\rho_s - \rho_w)/\rho_w$
- For placed blocks and block mats: $D_n = D$ = thickness of block, $\Delta_m = \Delta$,
- For gabions and mattresses: $D_n = d$ (= thickness of mattress [m]) and $\Delta_m = (1-n)\Delta$;
- The minimum thickness of mattress is equal to 1.8 D_n

In which D_{50} = 50% passing median stone diameter [m], M_{50} = 50% value of the mass distribution [kg]; n = porosity of stones [-], approx. n = 0.4.

The stability parameter φ_{sc} depends on type of application, some guide values are given in Table 8.3.

Table 8.3 Values of stability factor (Pilarczyk 1998)

Revetment Type	Stability factor φ_{sc}	
Cover layer	Continuous protection [-]	Exposed edges transitions [-]
Randomly placed, broken riprap and boulders	0.75	1.5
CC blocks, cubical shape, randomly placed in multi layer	0.80	1.50
CC blocks, cubical shape hand placed in single layer chess pattern	0.65	1.25
Riprap and placed blocks; Sand fill units	1.0	1.50
Block mats, gabions, washed-in blocks, geo-bags, concrete filled geo-bags and geo-mattresses, wire-mesh mattress	0.5	1.00
Gabions/ mattress filling by stones	0.75	1.5

The degree of turbulence can be taken into account with the turbulence factor K_τ , some guide values for K_τ are given in Table 8.4.

Table 8.4 Turbulence Intensity Factor K_τ (current) (FAP 21/22)

Turbulence Intensity	K_τ (-) Gabions, Mattresses	K_τ (-) 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
Jet impact, screw race velocity, hydraulic jump	3.0 - 4.0	3.0 - 4.0

With the depth parameter $K_{h,}$ the water depth is taken into account, which is necessary to translate the depth averaged flow velocity into the flow velocity just above the revetment. The depth parameter also depends on the measure of development of the flow profile and the roughness of the revetment. Logarithmic velocity profiles exist for long stretches with constant bed roughness. For most engineering works as bottom protection or slope protection, the non developed velocity profile is usually present. K_h is the coefficient due to conversion from the local mean bottom velocity to mean velocity ū.

The following formulas for K_h are recommended (Pilarczyk 2008):

Fully developed profile: $$K_h = 2 \cdot \left[\log\left(1 + \frac{12h}{k_s}\right) \right]^{-2} \tag{8.7}$$

Non-developed profile: $$K_h = \left(1 + \frac{h}{k_s}\right)^{-0.2} \tag{8.8}$$

Very rough flow ($h/k_s < 5$): $K_h = 1$ (8.9)

Where, h is the water depth (m), in the case of dimensioning the revetments on a slope, the water level at the toe of the slope must be used for h. k_s is the bed roughness (m) given approximately by: k_s = 0.1D for smooth units (i.e. pitched concrete blocks), k_s = D for block mats, and k_s = (1 to 3)D_n (rough units, i.e. rock). For riprap, k_s is equal usually to twice the nominal diameter of the stones ($k_s \approx 2D_n$), for bags it is usually equal to the thickness (d), for mattresses it depends on the type of mattress: k_s of about 0.05 m for smooth types and about the height of the rib for articulating mats.

Note: usually $12h/k_s$ is applied, however, by using $(1+12h/k_s)$ the discontinuity at small values of h can be avoided; the same adjustment is also applied for other velocity distributions. The effect of this additional (imaginary) depth practically vanishes for $h/k_s>2$ (after Pilarczyk 2008).

For critical Shield's parameter Ψ_{cr} the type of material can be taken into account. Some guide values are given in Table 8.5.

Table 8.5: Critical Shield's Parameter Ψ_{cr}

Revetment Type	Ψ (-)
Riprap, small bags	0.035
Placed blocks, geobags	0.05
Blockmats	0.07
Gabions	0.07 (to 0.10)
Geomattresses	0.07

Following formula have been reported in BWDB (1993)

(2) Isbash

$$W = \frac{4.1\times 10^{-5}\cdot S_s.V^5}{(S_s-1)^3\cdot\cos^3\alpha} \tag{8.10}$$

(3) California State Highways:

$$W = \frac{2\times 10^{-5}\cdot S_s\cdot V^6}{(S_s-1)^3\cdot\sin^3(70-\alpha)} \tag{8.11}$$

(4) PIANC

$$D_n = \frac{0.70V^2}{g(S_s-1)\cdot\cos\alpha\left[1-\left(\tan^2\alpha/\tan^2\theta\right)\right]^{0.5}} \tag{8.12}$$

(5) JMBA

$$D_n = \frac{0.7V^2}{2\cdot(S_s-1)\cdot g}\cdot\frac{2}{\log(6h/D)^2}\cdot\frac{1}{\left[1-(\sin\alpha/\sin\theta)^2\right]^{0.5}} \tag{8.13}$$

(fps or metric unit) (the equation has been developed for cc blocks)

In these equations:

W	(kg or lb)	weight of Individual stone
D	(ft or m)	diameter of stone
Dn	(ft or m)	dimension of cube
V	(ft/s or m/s)	mean velocity at the adjacent channel
h	(ft or m)	depth of water
S_s	(-)	specific gravity of stone
α	(°)	slope of bank
θ	(°)	angle of repose of revetment material
Ψ	(-)	Shield's parameter
g	(m/s^2 or ft/s^2)	gravitational acceleration

The PIANC equation which is stated to be a form of Isbash equation gives much lower size of revetment materials compared to other equations. The equation suggested by Pilarczyk and JMBA both considered depth factors for computation of size of revetment materials, but the Pilarczyk method also used turbulence intensity by an empirical coefficient. Considering all these factors it is recommended to use the Pilarczyk equation. However the designer may consult the other available equations for comparative analysis. Final selection will be based on the findings of the physical model results.

8.3.2 Stone Gabions and Mattresses

(1) For calculation of the required thickness of stone or brick filled mattress systems the relative density $(1-n)\Delta_m$, considering the volume of voids between the individual filling elements, shall be applied in place of Δ_m. The percentage of voids in the mattress fill can be estimated to:

$n = 0.40$	for stone filled mattresses
$n = 0.15$	for brick filled mattresses

- The minimum thickness of stone filled mattress should not be less than $1.8D_{50}$ (with $D_{50}=D_n/0.85$) or 15 cm.
- Required nominal diameter D_{50} or filling material can be calculated by Eq. 8.6, taking the respective stability coefficients for mattress filling (Δ_m, Φ_{sc}, Ψ_{cr}, θ)
- The minimum size of stones must be larger as compared to the width of the wire mesh sizes

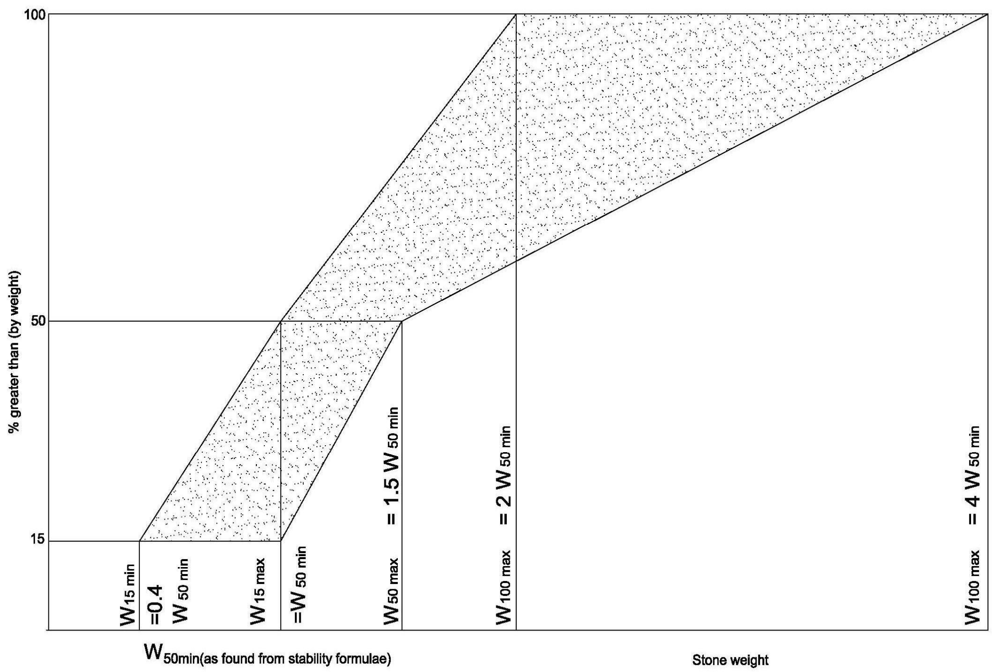


Fig. 8.3 Recommended grading range of stone filling for rip-rap and mattress system (after PIANC, 1987)

(2) The grain diameter for filling the mattress can also be determined by the following equation (PIANC 1987a):

$$d_{50} = h \cdot \left[\frac{u_{cr}}{B\sqrt{k' \Psi_{cr}\, g\, \Delta_m h}} \right]^{2.5} \qquad (8.14)$$

where,

d_{50}	(m)	grain diameter
h	(m)	water depth

k′	(-)	slope reduction factor= $[1-(\sin^2\alpha/\sin^2\theta)]$
u_{cr}	(m/s)	critical velocity
θ	(°)	angle of internal friction of material
Ψ_{cr}	(-)	critical Shield's parameter; = 0.1; in case of initiation of movement
Δ_m	(-)	relative density = $(\rho_s-\rho_w)/\rho_w$
ρ_s	(kg/m^3)	density of protection material
ρ_w	(kg/m^3)	density of water
B	(-)	co-efficient of flow condition = 5 to 6 for major turbulent flow, = 7 to 8 for normal turbulence, = 8 to 10 for uniform flow and minor turbulence

The thickness of the mattress t_m is related to the stone size and is given by:

$$t_m = 1.8d_{50} \tag{8.15}$$

(3) For estimation of thickness of geo-textile mattress the following formula can be applied:

$$d \geq \frac{\Phi \overline{u}^{\,2}}{\Delta_m \, 2g} * K_h * K_s \tag{8.16}$$

where;

d	(m)	thickness of mattress
K_h	(-)	depth factor
K_s	(-)	slope factor
g	(m/s^2)	acceleration due to gravity
Φ	(-)	= 0.8 for continuous protection, = 1.0 for exposed edges

8.4 Stability of Revetment under Wave Attack

For wave induced impacts on armouring units, several theoretical calculation methods are available for the design. Due to various input parameters involved in different methods a direct comparison is rather difficult. The Pilarczyk (1990) formula allows for calculation of different structure components and includes the breaker type specific dynamics of the wave impact by introducing breaker similarity index ξz.

The minimum dimensions for the stability of the cover material under wave attack can be determined by various formulas placed below. The Pilarczyk formula being more universal with its breaker similarity index is normally preferred.

(1) Pilarczyk, (1990)

$$D_n \geq \frac{H_s \cdot \xi_z^b}{\Delta_m \cdot \Psi_u \cdot \phi_{sw} \cdot \cos\alpha} \tag{8.17}$$

Where

Dn	[m]	characteristic size of the revetment cover layer (single unit size for loose elements, thickness for mattress systems)
Hs	[m]	significant wave height
Δ_m	[-]	relative density of submerged material = $(\rho_s-\rho_w)/\rho_w$
g	(m/s^2)	acceleration due to gravity (= 9.81)
$\varnothing_{sw}$	[-]	stability factor for wave loads
Ψ_u	[-]	system specific stability upgrading factor
α	[°]	bank normal slope angle
ξz	[-]	wave breaker similarity parameter= $\tan\alpha \cdot \frac{1.25 \cdot T_m}{\sqrt{H_s}}$
T_m	[s]	mean wave period
b	[-]	wave structure interaction coefficient, dependent on roughness and porosity of protective material

The formula is restricted to values ξz <3 and cotα ≥ 2, i.e. to plunging breakers, which generate high local pressure heads. Otherwise overestimation of the unit size is likely, because dynamics of the breaking process are diminishing.

In the formula 'b' is the exponent related to the interaction between waves and revetments (0.5 ≤ b ≤ 1.0). For rough and permeable revetments, b = 0.5, for smooth and less permeable placed-block revetments it is close to unity. For other systems b = 0.67 may be applied. Ψ_u is the system specific stability upgrading factor.

As for the design against current attack, the required thickness of stone or brick filled mattress systems shall be calculated on the basis of relative density $(1-n)\Delta_m$. Values for 'n' are given in section 8.3.2. The nominal dimension of fill material can also be computed by equations mentioned in section 8.3.1, taking the respective stability coefficients for mattress filling (Δ_m, φ_{sw}, Ψ_u) into account. Minimum thickness of mattress shall not be less than $1.8D_n$.

The material and armour layer unit specific coefficients to be applied for design against wave attack are summarized in Table.8.4. The coefficient for stability upgrading factor (Table 8.6) are based on mainly granular sub-layer.

Table 8.6: Coefficients for design of various cover materials against wave attack

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

(2) Hudson

$$W = \frac{H_s^{\,3} \cdot \rho \cdot \tan\alpha}{k \cdot \Delta_m^{\,3}} \tag{8.18}$$

(3) Iribarren

$$W = \frac{f \cdot H_s^{\,3} \cdot \rho}{\Delta_m^{\,3} (\cos\alpha - \sin\alpha)^3} \tag{8.19}$$

Where

W	(kg)	weight of revetment material
H_s	(m)	significant wave height
α	(°)	bank slope angle
f	(-)	coefficient related to the amplitude of the wave and slope angle
Δ_m	(-)	relative density of submerged material = $(\rho_s - \rho_w)/\rho_w$
ρ_s	(kg/m^3)	density of protection material
ρ_w	(kg/m^3)	density of water
k	(-)	coefficient varying from 3.2 for smooth quarry stone to 10 for tetrapods

8.5 Thickness and Grading of Riprap

8.5.1 Riprap Thickness

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

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

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

- Inglis (1949) recommended following formula to compute thickness of protection required on the slope of revetment,

$$t = 0.06\, Q^{1/3} \tag{8.20}$$

Where,

t	(m)	thickness of stone riprap
Q	(m^3/s)	discharge

The Inglis formula apparently gives excessive thickness for higher discharge.

- The thickness of stone pitching and soling for permanent slopes required at head, body and tail of guide bank for river flowing in alluvial plains as recommended by Gales (1938) is given in Table 8.7.

Table 8.7 Gale's Table to Compute Thickness of Stone on Slope

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

Note that Gales tables were outdated with the beginning of physical hydraulic model tests in the 1930s (undertaken by Inglis) and systematic observations in models and field.

- Spring (1903) recommended thickness of stone in inches for covering rough, heavy and loose stone for pitching from low water upwards as shown in Table 8.8.

Table 8.8 Spring's Table to Compute Thickness of Stones on Slope
(final required slope coverage)

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

The thickness suggested above should be increased by 50% when the riprap is placed under water to provide for uncertainties associated with the type of placement (US Army Corps of Engineers).

While the thickness of the cover layer and the size of the protection elements determines its stability to flow attack and provides certain safety margins, it does not necessarily mean that the cover layer is functional as filter layer towards fine underlying soils. This can only be assured through the provision of filter layers, or alternatively increasing the thickness of more uniform cover layers substantially.

8.5.2 Riprap Grading

Though the exact size distribution of a riprap mixture is not critical, but it should form a smooth grading curve without a large spread between minimum and maximum sizes. Stones should be reasonably well graded. Table 8.9 shows grading specifications for three classes of riprap which have been considered suitable for a fairly wide range of stream flow situations. (See Section 8.7.4)

Table 8.9: Suggested Stone Riprap Grading for Stream Bank Revetment (Source – Neill 1973, pp. 120)

Class-I Nominal 12 inch diameter or 80 lb weight Allowable local velocity up to 10 ft/sec	Grading Specifications 100% smaller than 18 inch or 300 lb At least 20% larger than 14 inch or 150 lb At least 50% larger than 12 inch or 80 lb At least 80% larger than 8 inch or 25 lb
Class-II Nominal 20 inch diameter or 400 lb weight Allowable local velocity up to 13 ft/sec	Grading Specifications 100% smaller than 30 inch or 1500 lb At least 20% larger than 24 inch or 700 lb At least 50% larger than 20 inch or 400 lb At least 80% larger than 12 inch or 70 lb
Class-III Nominal 30 inch diameter or 1500 lb weight Allowable local velocity up to 15 ft/sec	Grading Specifications 100% smaller than 48inch or 5000 lb At least 20% larger than 36 inch or 2500 lb At least 50% larger than 30 inch or 1500 lb At least 80% larger than 20 inch or 400 lb

8.6 Design of Filter Material

8.6.1 General

The most important part in a revetment with regard to critical seepage forces is the area where wave attack and piping occurs. Seepage forces are at maximum during the falling stages of the river. They may also occur at the toe of the revetment and turbulent flow may induce particle motion around the block protection. Besides, some of the soils encountered may be susceptible to downslope migration. In general piping and particle migration may be tolerable to some extent, as long as resultant deformations in the slope of revetment do not lead to instability of the slope in itself.

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

- To prevent migration of sub-soil particles out of the bank slope (Retention criteria) and

- To allow at the same time movement of water through the filter (Permeability criteria).

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

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

- Granular filters, made of loose, bounded or packed grains, and

- Fiber filters, made of synthetic or natural materials.

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

A complete understanding of the properties of soil adjacent to filter is essential to proper filter design. A summary of important soil properties for filter design are:

- Soil particle size distribution,
- Atterberg limits,
- Dispersion potential by double hydrometer,
- Hydraulic conductivity.

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

- Coefficient of uniformity, Cu, from particle size distribution: Cu = d_{60}/d_{10}
- Linear coefficient of uniformity, C´u from particle size distribution: C´u = $d'_{50}/d'_{0} = d'_{60}/d'_{10} = d'_{70}/d'_{20} = d'_{100}/d'_{50} = (d'_{100}/d'_{0})^{1/2}$
- Coefficient of curvature, C_c, from particle size distribution:

$$C_c = \frac{d_{30}/d_{10}}{d_{60}/d_{30}} = \frac{d_{30}^2}{d_{60} \times d_{10}} \tag{8.21}$$

- Hydraulic conductivity of soil, can be measured directly from permeability test; if testing is not possible, the hydraulic conductivity can be estimated from the particle size distribution. (Fig.8.4)

8.6.2 Granular Filter

The properties of a granular filter depend significantly on the particle size. The filter criteria relates to the grading of the filter to that of the subsoil. If the filter has got no drainage function, it is necessary to check for filter uniformity to ensure that internal migration of fines does not occur. Among others, Pilarczyk (1990) defined certain criteria, regarding the characteristic grain sizes of the subsoil D_s and the Filter D_f, for granular filter which has to meet the following requirements:

$D_{15f} < (4\, to\, 5) \cdot D_{85s}$	:stability criteria
$D_{15f} > (4\, to\, 5) \cdot D_{15s}$	:permeability criteria
$D_{50f} > (20\, to\, 25) \cdot D_{50s}$	:segregation criteria
$C_u = \frac{D_{50}}{D_{10}} < 10$	:no migration
$C_u = \frac{D_{50}}{D_{10}} > 20$	Susceptible to migration

with

C_u	(-)	coefficient of uniformity
D_x	(mm)	x% of the material is smaller than diameter D

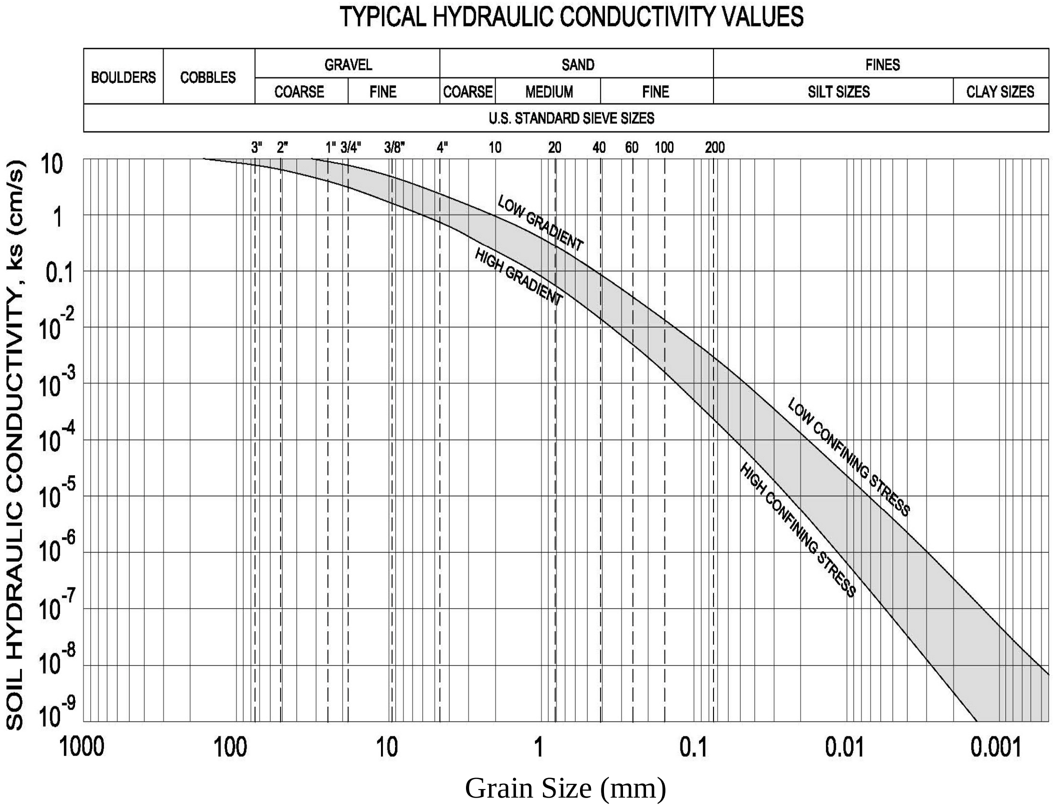


Fig.8.4 Characteristic Particle Size, d_{15} of Soil

It might be necessary to use more than one granular layer in order to achieve the required filter characteristic. In that case 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. Minimum overall thickness of a granular filter shall be 150 mm. Wherever practicable; each layer of granular material must be carefully compacted.

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

8.6.3 Fibre Filter

Fibre filter must have a porous structure where the combination of fibres and pores between them allows the soil particles to be retained in such a way that no blocking or clogging of the filter can occur. This is best achieved with a geotextile having the maximum amount of pores per unit volume allowing each soil particle size to find its appropriate place in the filter preventing at the same time further particles to pass through it.

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

Besides, the required functional characteristics of the geotextile in context with the existing subsoil properties, certain stability standards must be considered, which have to be defined with respect to planned use and construction technique to be employed. The criteria for geotextile filter selection are summarized below.

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

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

- Geo-synthetic drainage media
  - Volumetric porosity
  - Hydraulic transmissivity
  - Thickness
- Geotextile filter media
  - Apparent opening size
  - Permittivity,
  - Thickness

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

(a) Retention

- For soils with an uniformity coefficient less than 5: 0.05 mm< $O_{90} < 0.7\ d_{90}$
- For soils with uniformity coefficient greater than 5: 0.05 mm <$O_{90} < d_{90}$

(b) Permeability

$$\Psi = 5 \times 10^3 \times k_s \times i \qquad (8.22)$$

Where; Ψ	(l/sec)	permittivity
k_s	(m/sec)	permeability of the subsoil
I	(-)	hydraulic gradient in the soil

The PIANC (1987) design procedure involves the steps given in Fig. 8.5.

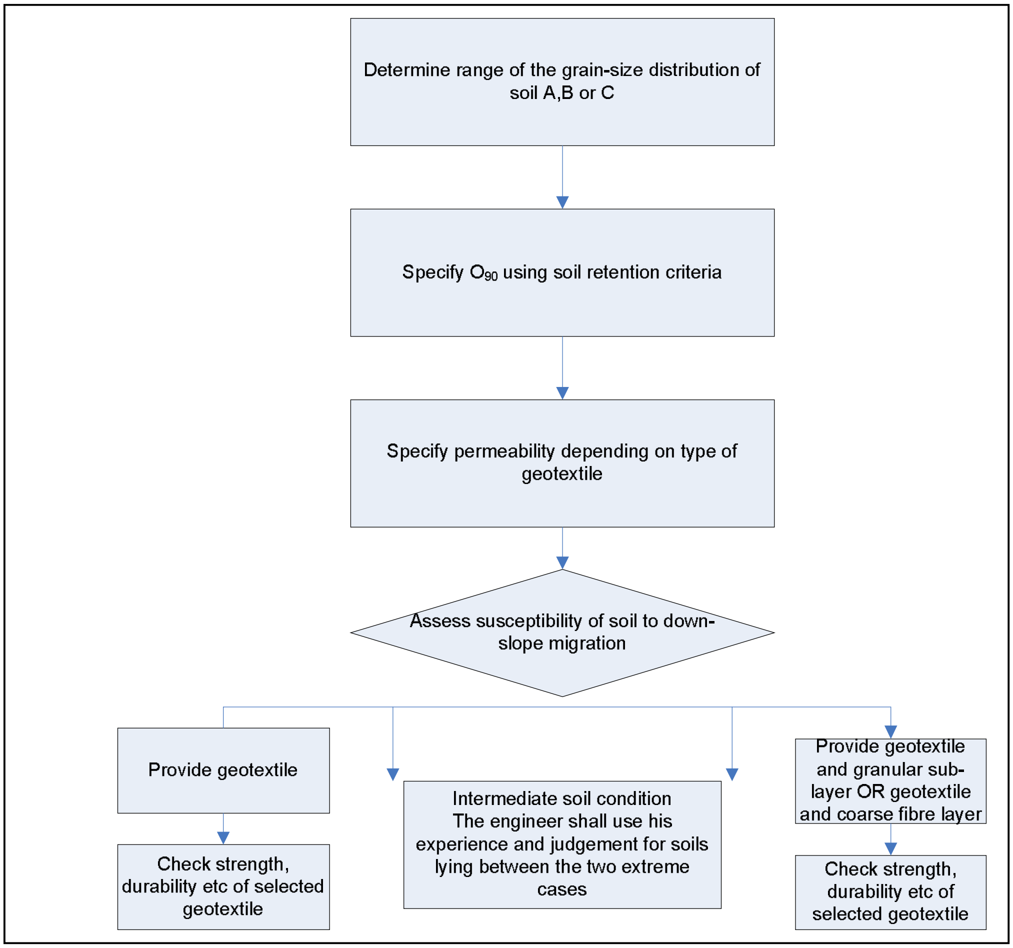


Fig 8.5 Design Procedure for geotextile filter

Various design steps are elaborated below:

Determination of the range of grain-size distribution

The grain-size distribution curve must be determined following international standard regulations, to allow for calculation of the various design parameters. As the filter characteristics of geotextile are mainly influenced by the fine compartment of the grain-size distribution, the PIANC method categorizes the soil by the screen fraction smaller than 0.06 mm grain size. Typical grain size distributions for different soil categories A,B and C are given in Fig. 8.6

Soil categories for geotextile filter design are classified as follows:

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

(2) Design for soil retention

The capacity of a geotextile in terms of soil retention is characterized by the effective opening size O_{90}, defined by the equivalent diameter of a grain fraction which is retained to 90% by the filter material in a sieving test. This value is normally available in product information sheet provided by the manufacturer. The minimum requirements for the geotextile filter to be considered for the dynamic load conditions, characterized by high turbulent flow and wave attack, are specified in Table 8.11. The given retention criteria are only applicable for O_{90} values determined by the wet sieving analysis as defined by Swiss Standard SN640550.

Table 8.11: Soil Retention Criteria (adopted from PIANC, 1987)

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

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

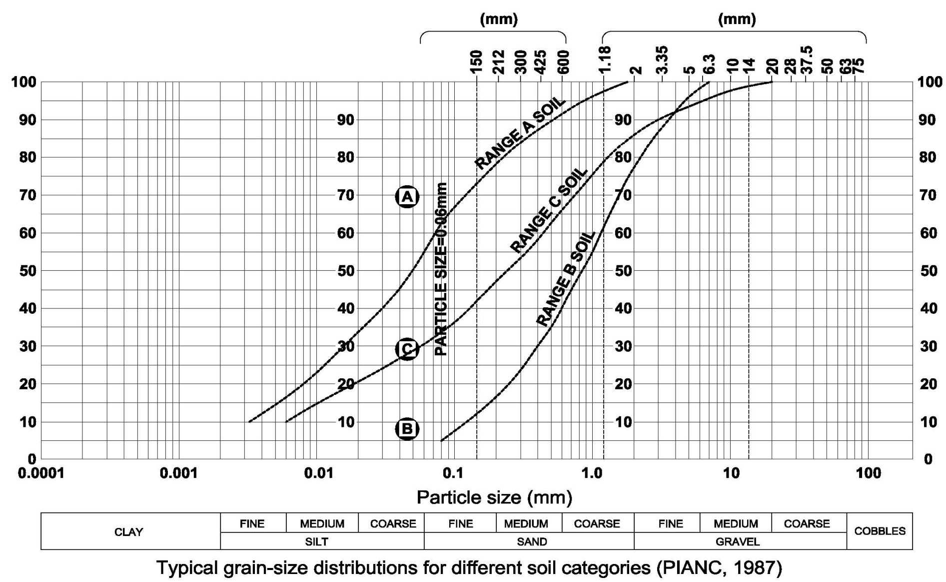


Fig.8.6 Typical grain size distributions for different soil categories.

(3) Design for permeability

The geo-textile filter must maintain a long term permeability equal to 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, thickness of the material and as well as on the grain structure of the soil. In general the permeability of the geo-textile material is acceptable if:

$$\eta \cdot k_g \geq k_s \tag{8.23}$$

where;

η (-) material specific reduction factor,

k_g (m/s) permeability of the geotextile,

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):

$$k_s = 0.0116 \cdot D_{10}^2 \tag{8.24}$$

The reduction factor η for needle punched and other non-woven fabrics thicker than 2 mm (measured at a normal stress of $2KN/mm^2$) is defined as a constant: $\eta = 1/50$

(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 (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. 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 $0.02 \text{ mm} < d < 0.1$ mm.
$Cu = d_{60}/d_{10} < 15$:	Coefficient of uniformity (conservative estimate based on practical experience; for larger Cu values normally a secondary filter will establish).
$Ip = W_L - W_p < 0.15$	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 (Wp). If Ip 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: $\frac{W_{(d<0.002mm)}}{W_{(0.002<d<0.06mm)}} < 0.05$

The above given conditions allow only for a rough estimation regarding the risk of down-slope migration process at certain sub-soil properties.

(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, which should be fine enough to provide an adequate damping effect and coarse enough to be retained by the cover layer. This type of measure is suggested in only the places where a permanent or very long term seepage source are active through a less permeable strata. A granular filter (Triangular shaped) below the geo-textile in the area having continuous seepage through very weak permeable strata may help to arrest downslope migration and clogging possibility of the geo-textile filter.

Furthermore, the surface of the soil underneath a geo-textile can be stabilized covering with a granular sub-layer (minimum 50 mm thick) to act as a cushion between the geo-textile filter and the core material.

Use of geo-textile material is the fastest way to install a filter, which is important in view of the restricted construction windows available in Bangladesh.

8.7 Design of Toe Protection

8.7.1 Toe Scour Estimation and Protection

Toe protection is required when currents and/or waves scour and undermine the toe of a bank or a structure which may possibly result in sliding of slopes. Lack of protection of the toe of the revetment against undermining is a frequent cause of failure of revetment. Therefore, protection of the toe of revetment by suitable method is a must. This is true not only for riprap, but also for a wide variety of protection techniques. The scour is the result of several factors including the factors mentioned below:

1. *Change in cross-section in meandering channel after a bank is protected:* after a bank is protected, the thalweg can move towards the outer bank and/or a channel with highly erodible bed and bank can experience significant scour along the toe of the new revetment.

2. *Scour at high flows in meandering channel:* Bed observed at low flows is not the same as that exists in high flows.

3. *Braided Channels:* Scour in a braided channel can reach a maximum at intermediate discharges where the flow in the channel braids concentrates along the protective work or attacks the banks at a sharp angle.

8.7.2 Toe Protection Methods of Revetment

Toe protection of revetments may be provided by following methods:

1. *Extension to maximum scour depth:* Lower extremity of revetment placed below expected scour depth or founded on non-erodible bed materials. Theses are preferred method, but can be difficult and expensive when underwater excavation is required.

2. *Placing launchable stone:* Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack. As the attack and the resulting erosion occur below the stone, the stone is undermined and rolls/slides down the slopes, forming a surface cover layer reducing the erosion. 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 launching apron is assumed to cover and stabilize the bank-sided river profile, reducing further erosion of the bank.

   Chapter 9 of **Inglis (1949)** defines falling (or launching) aprons as follows:

   A falling apron is a covering of loose stones laid on the berm of a river embankment to protect it from erosion. The stones, usually "one-man" stone weighing 80-120 lbs, are laid during low water in a horizontal "apron" on the sand or silt berm deposited by the river. Stone is also laid on the slope of the embankment, but this is intended to remain in position; whereas the "apron" is intended to "launch" or subside when the underlying sand is scoured by the river current – so as to form a continuous protection below water level. This method of protection appears to have been originated by James R. Bell and was described in his Paper of 1890 on the "Bell bund" system of river control at bridges. Sir Francis Spring produced a modified design, now in general use *in Northern India, in his "River Training and Control on the Guide Bank System" (1903).*

Joglekar (1971) adds:

"*A stone cover known as an apron is laid on the toe of the slope on the river bed so that scour undermines the apron first, starting at its farthest end and working backward towards the slope. The apron then launches to cover the face, with stone forming a continuous carpet below the permanent slopes. Adequate quantity of stone for the apron has to be provided to ensure complete protection of the whole scoured face. This quantity will depend on the apron thickness, depth of scour and slope of the launched apron".*

The method had been widely used on sand bed streams and often works as the attack is limited in time. The method, however, has two limitations: (i) for various reasons, launching apron as provided in the design, may not launch properly and the launched material may not provide adequate coverage against fine easily erodible subsoils, and (ii) launched slopes are less predictable if cohesive material is present, where the angle of internal friction is steeper than that of the launching materials.

Placement alternatives include:

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

A falling apron provides a satisfactory form of temporary bank protection where the material of the river bed or sea coast consist of sand or other coherent material; but it is quite unsuitable where the bed consists of material which does not scour freely- such as clay or rock. It can further be stated that an apron does not launch satisfactorily unless

(i) the angle of repose of the underlying material is flatter than that of the stones/cover material, and (ii) the material scours easily and evenly.

Owing to the flexibility of an apron, it can fail in the following ways, due to:

(i) launching taking place beyond the capacity of stone reserve.
(ii) slips and 'blow-outs' in the bank occurring due to steep subsoil water gradient resulting from a rapidly falling river, or abnormally deep breathing (fluctuating water levels) of several feet.
(iii) irregular disposition of stone on the slope due to the presence of layers of coherent material, such as clay.
(iv) a layer of coherent, or resistant, material becoming exposed at the toe, so that all stone reaching the toe is washed downstream until all available stone in the horizontally laid apron and in the original slope above water level is exhausted, leaving part of the bank material exposed to the current and wave action.

In Bangladesh protection is generally provided by flexible launching apron of cc blocks or boulders or rocks or gabions of galvanized wire mesh filled with bricks or boulders. Flexible launching aprons are generally preferred because they are less costly.

Since the flexible apron has to fit to the contours of the scour, their estimated depth is the basis for design rules. Boulders/rocks are assumed to launch to a slope of 1:2, whereas cc blocks/soil cement blocks or brick/stone filled gabions launch to a slope of about 1V:1.5H. This steeper slope is critical with respect to the geotechnical slope stability of soils commonly found along the banks of major rivers in Bangladesh and can result in local slope failures.

8.7.3 Dimension of Falling Apron

Among the various methods, launching apron or falling apron has been considered to be most economic and common method of toe protection of revetment so long its limitations are understood. Falling aprons are generally laid horizontally on flood plain/river bed at the foot of the revetment, so that when scour occurs, the material will launch and will cover the surface of the scour hole in a natural slope. Historically, starting from the work of Spring and Gales the launched quantity was computed assuming a launched apron thickness similar to the thickness of pitching work above water. The quantities were calculated as geometrical area depending on launched thickness, depth of scour, and slope of the launching apron. This approach was further substantiated by the results of the systematic model tests published by Ingles in 1949 and follow on model tests conducted in Bangladesh during the Flood Action Plan in the early 1990s.

a) Thickness and shape of Launching Apron

Regarding the best disposition of stone in an apron, Spring stated that, 'the apron should be made broad enough and thick enough not only to cover up the erodible matter exposed by sub-surface erosion; but also to keep the deep part of the erosion so far off, that the automatically pitched sub-surface slope may have an inclination of fully two to one'.

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

Since apron stone shall have to be laid mostly under water and cannot be hand placed, thickness of apron at junction of toe and apron according to Rao (1946, after Varma, Saxena and Rao 1989) should be 1.5 times the thickness of riprap in slope. Thickness at river end of apron in such case shall be 2.25 times the thickness of riprap in slope.

The slope of the launched apron was suggested by Spring and Gales as 1:2 and according to Joglekar (1971) it should not be steeper, but also not flatter than 1:3. *Different shapes and dimensions were suggested by the above mentioned authors for the apron to be placed in the river bed expecting/estimating a thickness of the launched apron as about 1.25 times the thickness of the slope cover layer*. The face slope of the launching apron may be taken as 2:1 for loose stone as suggested by Spring (1903) and Gales (1938). Details are shown in Fig. 8.7a, Fig. 8.7b. and Fig. 8.7c.

The early authors (Spring, Gales) realized the need for multiple layer coverage of great thickness under water. Due to technical constraints, they relied on falling aprons, placed above low water level to achieve the desired protective coverage of the underwater slope. Due to lack of field observations and model tests, the actual falling or launching behaviour of the materials could not be ascertained. Experiments conducted by Inglis on launching behaviour of falling apron on slope and published in 1949 shows no evidence of multiple layer thickness coverage after launching. However his experiments provided indicative results of the launching behavior of falling aprons underwater (Figure 8.8). Inglis concept and formulas are quite popular in Indian subcontinent.

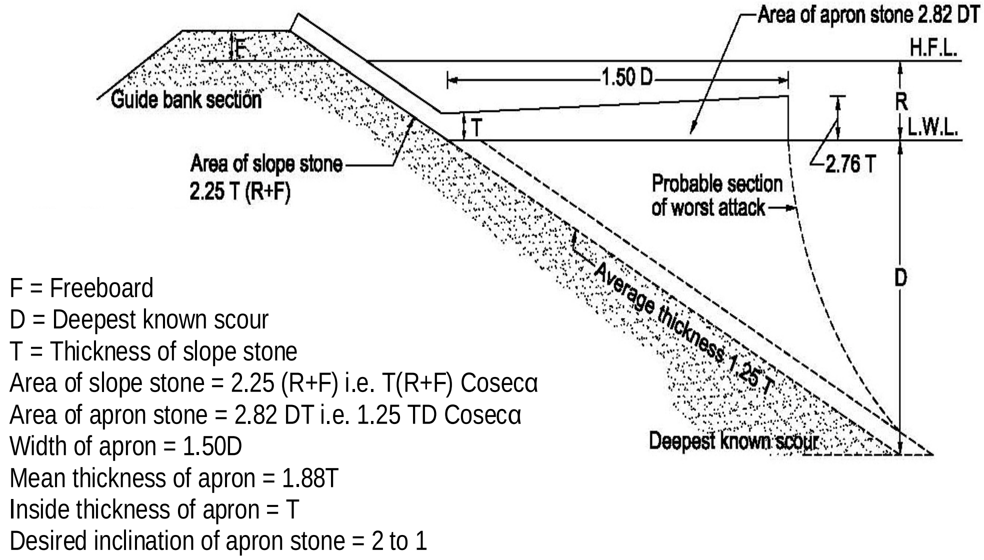


Fig.8.7a Shape of apron suggested by Spring (1903)

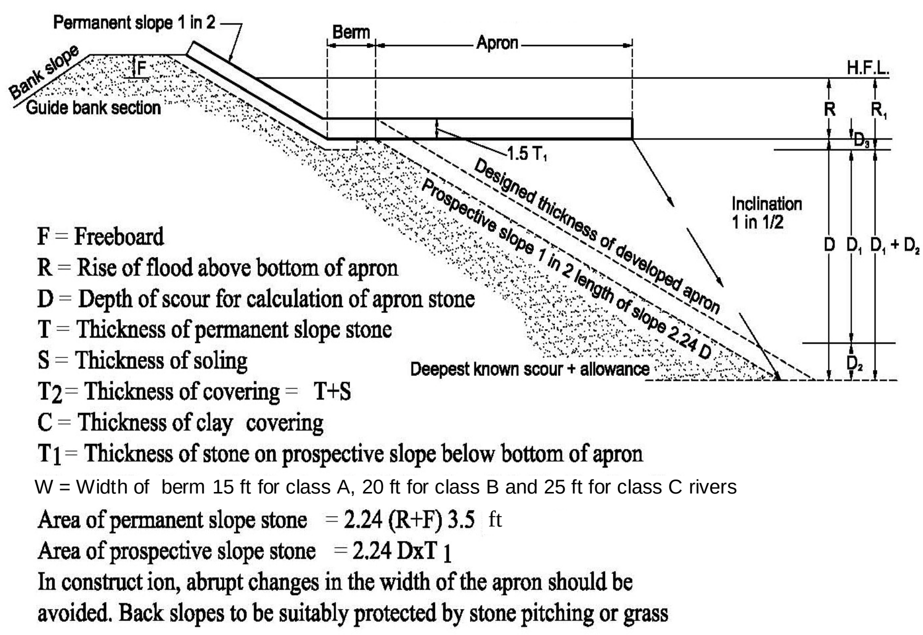


Fig.8.7b Shape of apron suggested by Gales(1938)

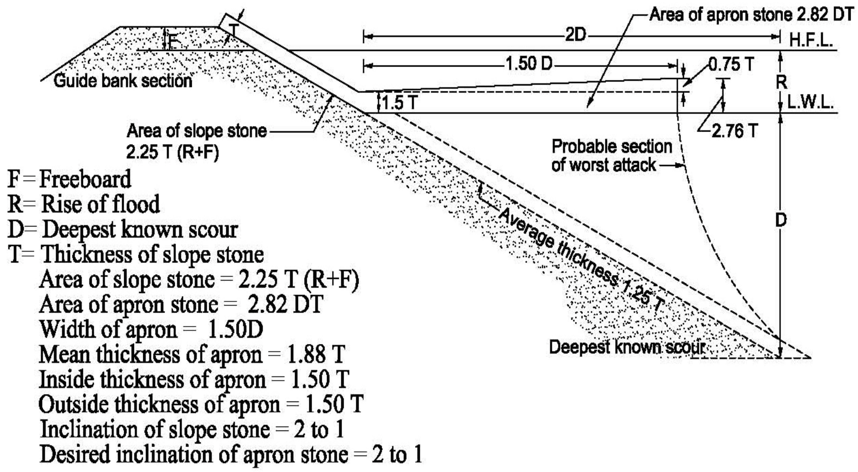


Fig.8.7c Shape of apron suggested by Rao(1946)

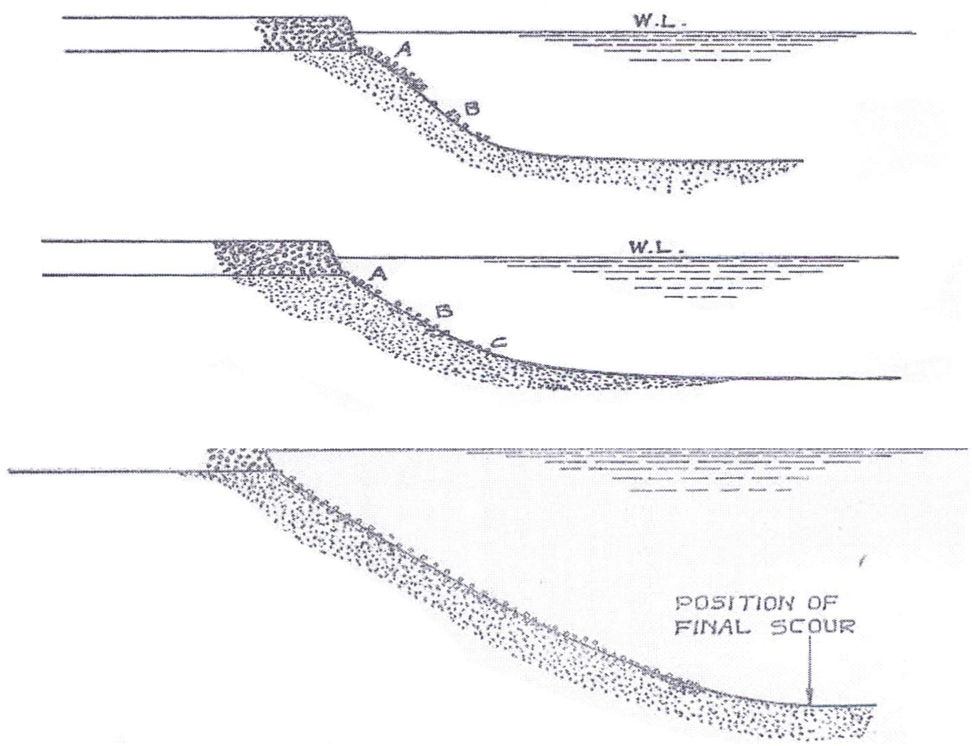


Figure 8.8: Sequential launching of falling apron stone (Inglis, 1949)

Inglish further supported the observation of single layer launching, dating from earlier model tests in 1937 and reported in correspondence with Gales on the failure of the Harding Bridge guide bund in 1938:

"…the … idea that the thickness of the layer of stone remaining on a slope after an apron launched could be regulated by the distribution of stone in the apron had been shown by experiments to be incorrect. ... If a thicker apron were considered desirable, extra stone would have to be dropped from barges."

Physical model tests conducted under different components of the Flood Action Plan (FAP) in the early 1990s confirmed Inglis' observations. FAP 1 found single layer coverage with concrete blocks, which was later confirmed by FAP 21, another component (Fig. 8.9). Halcrow, FAP 1 observed: "About one layer of blocks covered the eroded slope below the apron setting level".

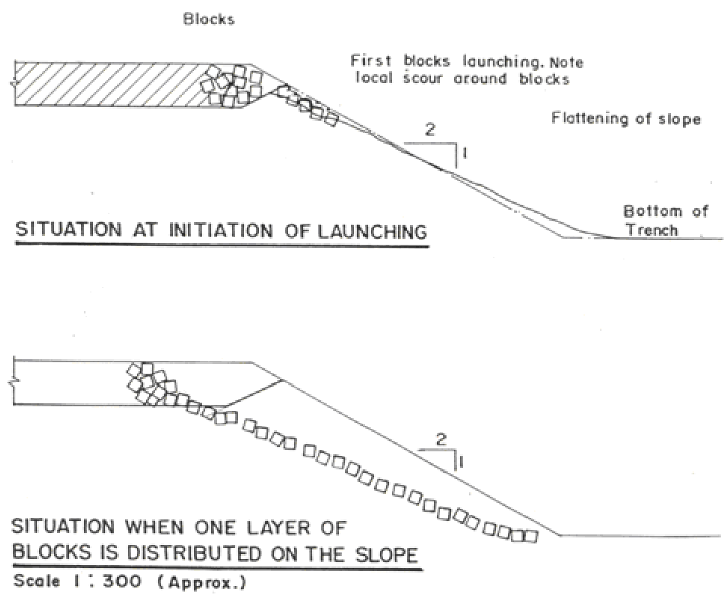


Figure 8.9: Single layer coverage after launching of concrete blocks (Halcrow, FAP 1, later Riverbank Protection Project, 1993)

To avoid the problem of the single layer coverage, the provision of filter layers becomes essential in the case of the fine, non-cohesive soils in Bangladesh. The single layer coverage does not retain the fine subsoil, which is washed through the gaps (Fig. 8.10). This is called winnowing. The smaller and more angular the elements of the cover layer are, the bigger is the loss from winnowing. Larger and flexible elements reduce the winnowing, due to the smaller number of gaps. Multiple layers of protection are therefore the way to achieve sustainable protection. Flexible elements may serve as better alternative.

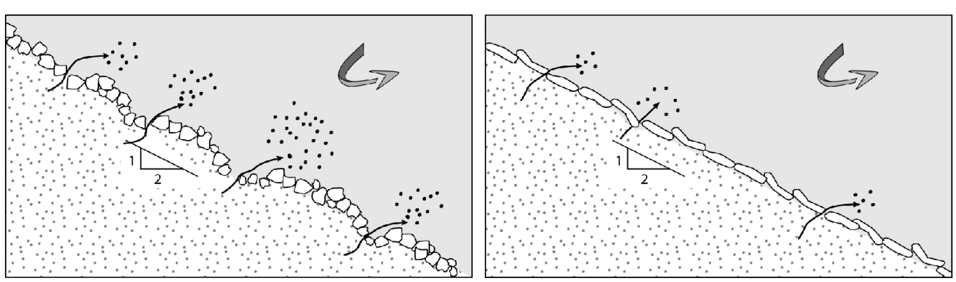
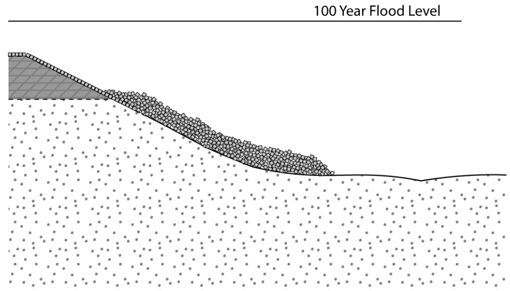


Figure 8.10 Winnowing or loss of fines through a single layer thick coverage

Despite the knowledge that falling aprons may not provide multiple layer coverage after launching in the field, formulas from Spring and Gales or modified versions based on a desired thickness after launching are still in use for their dimensioning. Generally assuming multiple layers after launching provides a comfortable safety margin but results in costly structures. Figure 8.11 shows the situation before and after launching of placed underwater protection, following common principles in use in Bangladesh.



Initially placed protection

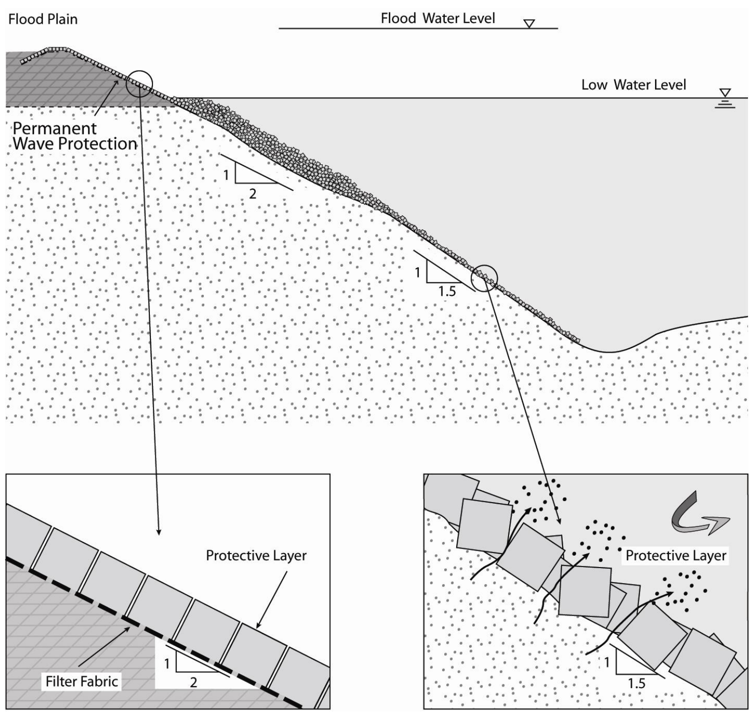


Fig. 8.11 Initially placed protection and insufficiently protected steep underwater slope after launching, susceptible to winnowing (loss of sand through gaps between protective elements).

b) Size of Apron Elements

The required size of protective elements for launching apron may be the same as the size of stone in slope revetment considering stream velocity as governing factor. It needs to be kept in mind that model tests show that graded hard material (rock) performed slightly better than uniform material. This is attributed to the fact that some finer elements reduce the voids between the bigger elements and lead to less winnowing losses. In case of flexible materials, such as geobags, the difference is less obvious.

c) Length of Launching Apron

Inglis notes that wide falling aprons provide better protection than short and compact ones. The general practice as recommended by Inglis (1949) is to lay the apron over a length of 1.5D, where D is the design scour below the position of laying. According to Hemphill/Bramley (1989) the length of the apron before launching should be designed to about 1.7*D, where D is the deepest scour. FAP 21 (2001) proposes a width of the falling apron between 70 to 100% of the scour depths. Experience with geobags indicates that wide falling aprons perform better and for this reason the falling apron used under JMREMP was 15 m wide, consisting of three layers of geobags.

d) Quantity of Stone in Apron

The designer faces a difficult task when dimensioning falling aprons. The reason being an unclear understanding about the scour depth. In line with the launching behavior the quantities of a falling aprons needs to be sufficient for single layer coverage to the deepest scour level. Prudence indicates adding some contingencies for locally deeper scouring. This means that the common formula can be applied with some minor modifications: T, the thickness of the layer after launching, is understood as the thickness of one element (D_n).

The standard design of the launching apron is based on the principle that protective elements of thickness D_n launch to the deepest scour depth D at a slope of 1V:2H.

- The length of launched apron is $= 2.25D$ and
- With a safety margin of 50%, the volume of launched elements per unit length is $2.25D.1.5D_n = 3.35D.D_n$.

The single layer slope coverage of launched falling aprons provides the initial protection. While this layer can reduce erosion rates temporarily, it needs to be upgraded to multi-layer protective systems by systematic dropping from the burge/boat, if long-term stable protection is desired.

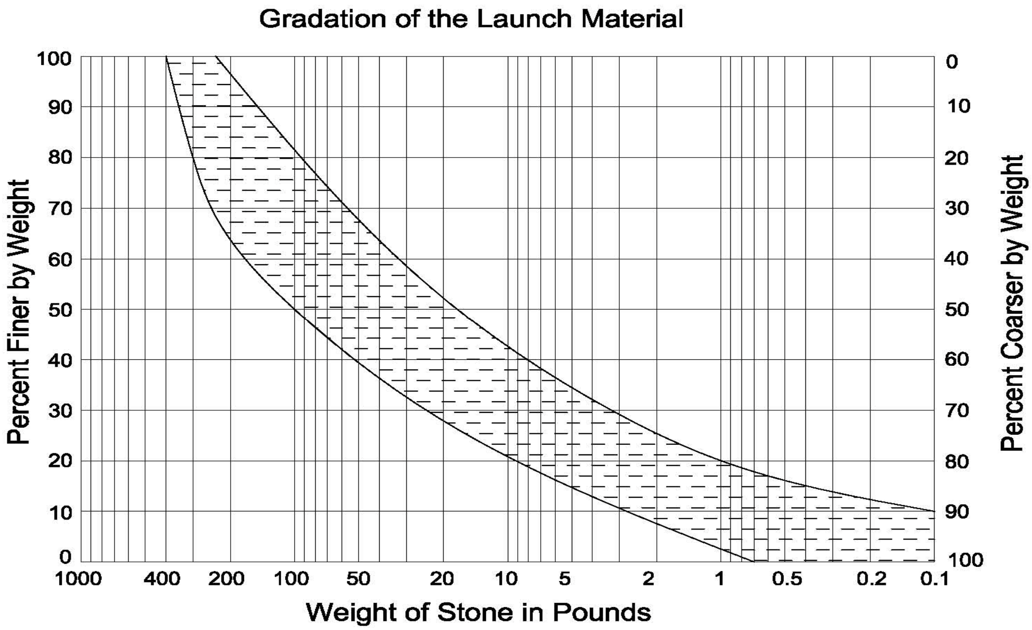
f) Location of falling apron

Falling aprons are ideally placed deep under water along the toe of the revetment (weighted riprap toes). This serves three purposes, (i) the launching is limited to the depth of the toe scour and not to provide coverage for the whole underwater slope as in the case of window or trench fill revetments (chapter 8.7.2), (ii) the soil at the toe and below is more consolidated than that of the higher strata and provides higher slope stability and more stable slopes after launching, and (iii) existing slopes of the upper part protected through systematic dropping of protection elements keep the scour hole away from the bank and reduce the risk of geotechnical slope failure.

8.7.4 Gradation of Launching Material

The most critical criteria for this type of structure is the gradation of launch material. The gradation is shown (hatched area) in Fig. 8.12, and was developed by numerous years of prototype experimentation in U.S.A. It has proved effective on numerous river systems under varying hydraulic conditions with velocities in excess of 15.0 ft/sec (4.6 m/sec). Any material that grades within the hatched area is acceptable for use as launch material.

It should be noted that this material has a large percentage of relatively fine particle, where up to 20% can weigh 1 lb (0.45 kg) or less. This fine material is thought to aid in the launch process and to provide a filter after launching. The design concept using graded materials (Fig. 8.12) is applicable for quarry rocks, natural boulders, CC blocks etc.



Stone weight pounds	Cumulative % Finer by weight
400	100
250	70-100
100	60-80
30	32-58
5	15-34
1	2-20
less than 0.5″ max.dimension	0-10

Note: 5% of the material can weight more than 400 pounds. However no piece shall weight more than 500 pounds

Gradation

Graded Stone C

Fig. 8.12 Gradation of the Launch Material

8.8 Innovative Revetments

8.8.1 General

The use of revetments, such as riprap, blocks and block mats, various mattresses, and asphalt in civil engineering practice is very common. The granular filters, and more recently the geotextiles, are more or less standard components of the revetment structures.

In general, based on experience, sand-filled elements can be used as temporary structures, or as permanent structures at locations with relatively low wave attack (H < 1.5m), or as submerged structures where direct wave forces are reduced.

Research on the stability of rock and block revetments reveals that, much knowledge has been developed about the possible failure mechanisms and methodology on development of stability criteria under current and wave load (see CUR/RWS, 1995a,b, Pilarczyk, 1998). Similar design methodology for block revetments has recently been extended in applicability by means of a desk-study for a number of geosystems, such as sandbags and sand- and mortar-filled mattresses. Also other stability aspects, such as soil-mechanical stability and residual strength were taken into consideration.

The main obstacle in application of geosystems is the lack of proper design criteria. Until recently, no or unsatisfactory design tools were available for a number of types of revetment and geosystems. From the literature review it can be concluded that the stability of the coastal structures composed of geosystems (bags, geotubes, geocontainers) can usually be expressed in a similar way as for rock, namely in terms of the k_D factor in Hudson's formula or in terms of the parameter $H/\Delta D_n$

The stability criteria for sand- and mortar-filled bags and mattresses have been made available in Klein Breteler, 1998, Stoutjesdijk, 1998, DELFT HYDRAULICS / DELFT GEOTECHNICS, 1998 and K.W. Pilarczyk 2000.

8.8.2 General Stability Criteria

(a) Wave-load Stability

There are two practical design methods available: the black-box model and the analytical model. In both cases, the final form of the design method can be presented as a critical relation of the load compared to strength, depending on the type of wave attack:

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = function\,of\ \xi_{op} \tag{8.25}$$

For revetments, the basic form of this relation is:

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \frac{F}{\xi_{op}^{2/3}}\ with\ \max imum \left(\frac{H_s}{\Delta D}\right)_{cr} = 8.0 \tag{8.26}$$

In which: F = revetment (stability) constant (-), H_s = (local) significant wave height (m), Δ = relative density (-), D = thickness of the top layer (m), and ξ_{op} = breaker parameter (-).

The relative density is defined as follows:

$$\Delta = (\rho_s - \rho_w)/\rho_w \tag{8.27}$$

with: ρ_s = density of the protection material and ρ_w = density of water (kg/m^3). For porous top layers, such as sand mattresses and gabions, the relative density of the top layer must be determined, including the water-filled pores:

$$\Delta_t = (1-n)\,\Delta \tag{8.28}$$

In which: Δ_t = relative density including pores (-) and n = porosity of the top layer material (-).

The breaker parameter is defined as follows:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{\frac{H_s}{L_{op}}}} \tag{8.29}$$

The wave steepness S_{op} is defined as:

$$S_{op} = \frac{H_s}{L_o} = \frac{2\pi\ H_s}{g T_p^2} \tag{8.30}$$

In which

$$L_{op} = \frac{g}{2\pi} T_p^2 \tag{8.31}$$

and α = slope angle (°), L_{op} = deep-water wavelength at the peak period (m), and T_p = wave period at the peak of the spectrum (s).

The advantage of this black-box design formula is its simplicity. The disadvantage, however, is that the value of F is known only very roughly for many types of structures.

The analytical model is based on the theory for placed stone revetments on a granular filter (CUR/RWS, 1995a). In this calculation model, a large number of physical aspects are taken into account. In short, in the analytical model nearly all physical parameters that are relevant to the stability have been incorporated in the "leakage length" factor.

The basic formulas of the analytical model are not repeated here. For these, reader is referred to (CUR/RWS, 1995a).

(b) Flow-load Stability

Severe flow attack may in practice occur on revetments, such as with flow over a steep slope and flow attack near many kinds of structures (downstream of sills, gates, discharge structures and the like). At these structures, the flow is often specifically determined by the geometry and the boundary conditions. With flow over a steep slope, such as on the downstream slope of a over-flow dam or dike, the situation is less ambiguous.

There are two possible approaches for determining the stability of revetment material under flow attack. The most suitable approach depends on the type of load that is whether the discharge or the flow velocity can be determined most accurately.

When the flow velocity is well known, or can be calculated reasonably accurately, Pilarczyk's relation (Pilarczyk, 1990) is applicable: (see also para 8.3.1(1) and equation 8.6)

The advantage of this general design formula of Pilarczyk is that it can be applied in numerous situations. The disadvantage is that the distribution in results, as a result of the large margin in parameters, can be rather wide.

With a downward flow along a steep slope it is difficult to determine or predict the flow velocity exactly, because the flow is very irregular (high turbulence, inclusion of air as a result of which the water level cannot be determined very well, etcetera). One is confronted with this when dimensioning the revetment of (the crest and) the inner slope of a dike in the case of flooding. In that case a design formula based on the discharge is preferable.

8.8.3 Properties of Geosynthetic Material/Geotextiles

Geosynthetic containers are multipurpose elements that can be manufactured according to almost any demand. That makes the geotextile fabrics and bags attractive for river bank protection. The functions of geo-textile used with soils are reinforcement, filtration, drainage, separation and protection. Geotextiles have to maintain these functions over many years. That's why the stability and durability of geotextile structure are very important.

The primary objective of a riverbank revetment is to stop the erosion. To do so the applied measure must retain particles of the soil.

(i) Durability of Geotextiles

Apart from the above properties the geotextile must last for a long period of time. Important criteria for longevity are resistance against abrasion, UV light and soil environment conditions like chemical or biological degradation. (*US Department of Transportation, 1998*). The life expectancy varies from author to author, going from minimum 35 years up to 110 years. The life expectancy according to different authors is shown below:

Table 8.12 Experience on life expectancy of geotextiles

Expectancy	Properties and exposure	Reference
> 100 years	Summary of geotextile properties for hydraulic and coastal engineering	Pilarczyk 2000
30 up to 110 years	Long time behavior of geotextile used in landfills	Muller Rochholz 2002
50 years	Non-woven geotextile bags in the revetments	Van De Burg, A, 2002
>35 years	Non-woven geotextile in dyke revetments as filters at the 'Mittellandkanal' in Germany; (The needle punched geotextile did not show weakness or problems after 35 years)	Heerten & Saathoff, 2004

8.8.4 Study on Geo-bag as an Alternative Launch Material

An economical source of stone is not available in Bangladesh and concrete blocks are also expensive. Geo-bags filled with sand can be made available at a much cheaper cost. Therefore, a graded geo-bag as launching material was developed and tested in Jamuna Meghna River Erosion Mitigation Project (JMREMP). The observation and monitoring on protection works undertaken by different size mix of geo-bags in two subprojects of JMREMP (PIRDP and MDIP) and Physical Model Study conducted in Vancouver, Canada on performances of geo-bags as launching material and comparison of their behaviour with other traditional hard materials has an indication towards using geo-bag as an alternative apron material. It is further gathered that the geo-bags used in spurs constructed in Mississippi river have launched successfully. Observations made on protection work (geo-bag apron) executed in PIRDP and MDIP also indicate that geo-bags can be used as launching material.

In JMREMP different configuration of apron was proposed and tested in the field. Similar configurations of apron were also tested in the Physical Model in Vancouver, Canada (NHC, 2006). The materials for launching were placed in four different configurations tested in model are shown in Fig. 8.13. For Configurations 1, 2 and 3 the launching material was placed at the foot of a slope protected with the same material.

Both prototype observation and model results confirmed that the areal coverage with a falling apron at the end (configuration-3 of the model study) as practiced in JMREMP

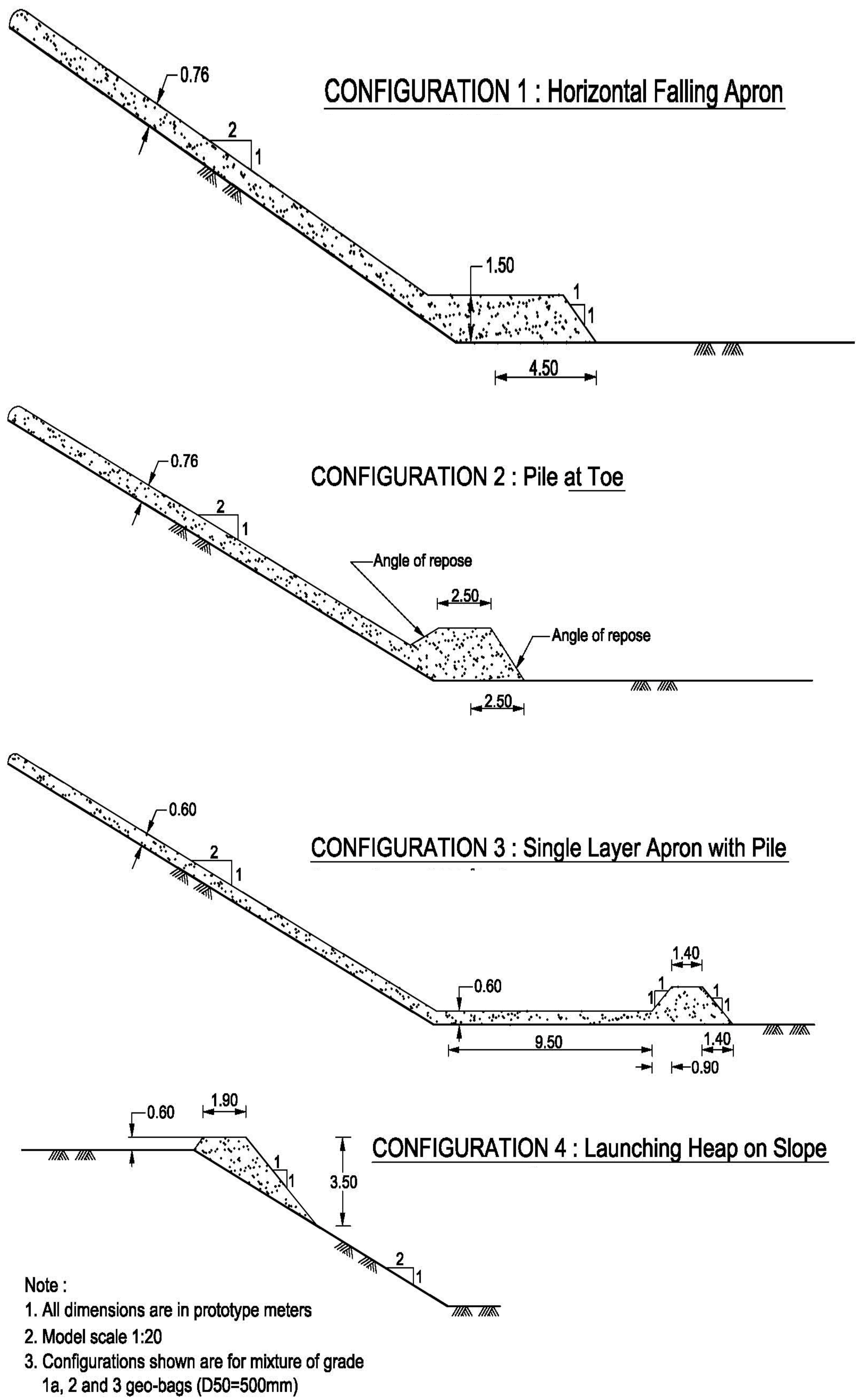


Fig.8.13 Launching Apron Configuration tested in laboratory model(NHC, 2006)

performed better in launching than other types of apron. Underwater inspection by divers and the post work section supported same observation.

Launch tests: The model examined the behaviour of various protective materials when placed in different ‘apron’ configuration and exposed to scour. Rock rip-rap was found to perform the best as a launching apron, and geobags were found to be superior to a single size concrete blocks as tested. Although recommendation of model test was to conduct more comparative test of graded CC block versus geobags. Of various apron configurations tested, an extended horizontal apron (configuration-3 of the model) performed the best. The extended (longer) horizontal apron with small pile at outer end produced more uniform launching with no vertical face and kept scour further away from the bank.

Incipient motion:

(1) The USACE equation can be adapted for predicting geo-bag incipient-motion velocities, assuming a characteristic size D equal to the cube root of length x width x thickness, and a shape factor C_s of 0.77.

(2) Figure 8.14 can be used for estimating relationships between incipient motion velocity and geo-bag size. A nominal D_{50} value, equal to cube root of as-filled length x width x thickness, should be inserted. The submerged, saturated relative density (s-1) of full-size bags, as determined from field tests, should be used in the equation.

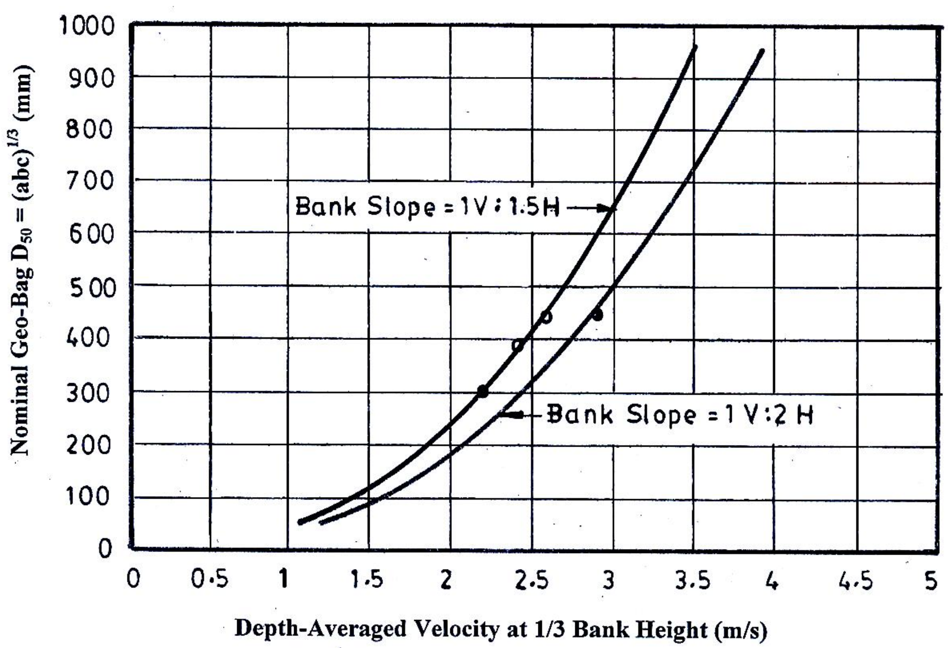


Fig. 8.14 Incipient motion for geo-bags of different size

The tests further indicated that

(1) The bags of 126 kg weight are stable under high flow velocities. At 4.5 m/sec velocity only a few bags were displaced from the test section even though a number of bags were locally displaced or flipped over they continued to provide protection.

(2) Shorter and more square bags performed better than longer bags.

(3) Thin and wide falling aprons of bags provide better protection than short thick ones.

(4) Geo-bags are displaced at lower velocities than riprap stones of similar weight – as would be expected because of their lower submerged density.

The amount of material is computed by estimating for the maximum scour and assuming 1V:2H launch slope. Assuming a cover thickness of about 60 cm, the amount of material launched in 1V:2H slope can be calculated on selected apron configuration.

Following notes have been referred to Fig: 8.14:

1. A proposed equation for the estimating the depth-averaged velocity at which a particular bag size just remains stable is:

$$V = K_1^{0.5}\, g^{05} (s-1)^{0.5}\, D_{50}^{0.4}\, Y^{0.1} / (C_s C_v)^{0.4} \qquad (8.32)$$

   Where V= Depth-averaged velocity at 1/3 Bank Height

   K_1= Side slope correction factor (=0.71 for 1V: 1.5H and =0.88 for 1V:2H)

   g = gravitational acceleration

   (s-1) = Submerged relative density of geo-bags (taken as 1.1 in model)

   D_{50} = $(abc)^{1/3}$, where a = Length, b = width and c = thickness of field geo-bag

   Y = Depth at 1/3 bank height

   C_s= Shape coefficient, proposed value = 0.77

   C_v= Coefficient for vertical velocity distribution, ranging from 1.0 to 1.28 for straight channels to abrupt bends (=1.1 in model).

2. For design applications, an adequate factor of safety should be applied to the bag size determined from the equation.

3. Extrapolation outside the test range should be done with caution. Further testing is recommended to determine applicability of graph to geobags of size, shape and mater different from those tested.

Incipient motion for different protection elements as observed in the model test at Vancouver, Canada are shown in Table 8.13:

Table 8.13 Summary of incipient motion for different protection elements as observed in the model test

Material	Size, D_{50} (mm)[1]	Dry weight (kg)	Side slope 1V:	V (m/s) [2] 1/3 bank	End of test [3](after exposure to 4.5 m/s)
Angular rock	250	22	1.5H	2.70	Approx. 25 rocks displaced
Rounded rock	330	50	1.5H	2.60	Approx. 15 rocks displaced
Concrete Blocks	300	65	1.5H	2.60	Approx. 30 blocks displaced
Geo-bag:A	440	126	1.5H	2.60	10 bags displaced
Geo-bag:B	390	90	1.5H	2.40	25 bags displaced
Angular rock	250	22	2.0H	3.50	20 rocks displaced
Rounded rock	330	50	2.0H	3.50	10 rocks displaced
Geo-bag:A	440	126	2.0H	2.90	5 bags displaced

[1]. D_{50} of geobags calculated as $(abc)^{1/3}$, where a=length, b= width, and c=thickness, as filled,
[2]. The velocity at which 10 elements had moved,
[3]. Criterion for incipient motion was flipping over of 10 bags. At the end of test 10 bags had left the test section even though many of the rest had flipped over or moved within the test area they still provided protection to the area.

8.8.5 Observations made on use of geo-bags (PIRDP and MDIP)

Geo-bags are found to be stable protection elements. They seem to provide a flexible cover layer with adequate resistance to flow forces acting on the revetment. In addition they contain filter characteristics and retain the fine underlying subsoil. Uniform sized geo-bags launch, in general forming a one-bag thick cover layer.

The geotextile bags used must withstand the hydraulic load of the current which can reach up to or a slightly higher than 3.0 m/s.

Geotextile bags were filled up to 100%. As a result of the compaction under water the density of sand changes from 1500 kg/m^3 to 1750 kg/m^3.

The final submerged weight of the filled bag (A-type), as observed, is between 62 kg and 70 kg.

Sand filled in geotextile bags (A-type) should be 126 kg, free of plastic, roots and other organic materials. It should not contain more than 5% silt and clay to the extent passing the 0.075mm sieve. (Stevens, M.A. 2006).

Table 8.14: Volume and Dimensions of Geobags used in JMREMP.

	126 kg; A-type	**78 kg; B-type**
Size of empty bag	1.03 m x 0.7 m	0.83 m x 0.6 m
Area of empty bag	0.72 m^2	0.50 m^2
Area of fully loaded bag (75% of empty bag)	0.54 m^2	0.037 m^2
Volume of locally dredged sand (1500kg/m^3)	0.084 m^3	0.052 m^3
Volume of compacted sand under water (1750 kg/m^3)	0.072 m^3 (86% of loose sand)	0.045 m^3 (86% of loose sand)
Computed thickness of filled bag at 1750 kg/m^3	16 cm	14 cm

The application of geobags in the context of adaptive riverbank erosion management is given in Appendix B. The main criteria used are:

Hydraulic design

- Waves (Pilarczyk)
- Current attack (Pilarczyk or modified USACoE equation 1991 based on Vancouver model – 126 kg geobags are stable for more than 3 m/s depth averaged flow)

Geotechnical design

- Geotechnical slope stability based on subsoil investigation
- Scour depth and falling apron, experience with Jamuna (max. additional scour: 15 m in 100 days)

Structural considerations

- Areal coverage (average 3 layers equals 6 bags 126 kg or 9 bags 78 kg per square meter)
- Systematic underwater construction (bank protection and falling aprons) based on actual situation at time of construction

8.8.6 Worked-out Design Example

Typical worked-out examples of design of a bank protection work are given in Appendix-B.

CHAPTER NINE

Design of Groynes

9.1 General

Groynes are stone, gravel, rock, earth or pile structures built at an angle to the river bank with a view to deflect the flowing water away from critical zones to prevent erosion of bank. Groynes vary considerably not only in their construction and appearance, but also in their action on stream flow. 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 vulnerable banks and provide safety against 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. Groynes can be classified as permeable and impermeable.

9.2 Selection of Types of Groynes

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

- A single or a series of groynes
- A permeable or a impermeable groyne, or
- A submerged or a non-submerged groyne

Groyne can be used as a single one or in series. Groynes in series are used where a long reach of bank needs to be protected. The flow pattern between groynes built in series can be influenced by an optimized layout plan. Several permeable groynes placed in a series have the advantage of reduction of the flow velocity near the bank on properly selected spacing of the groynes.

Single groyne protects the bank downstream of the groyne over a relatively short distance against bank erosion. Due to its sensitivity to change direction of flow attack, single groyne is not recommended for general application.

9.3 Length and Spacing of Groyne

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

Often the length of the groyne is kept longer than $2.5d_s$ where d_s is the scour depth below the bed level, to avoid bank erosion at u/s and d/s of the groyne. Normally groyne longer than one-fifth of the river width is not provided (Varma et al 1989).

Experience with groynes in Indian rivers show that a blockage of about 25% of the channel width on bankfull stage can be used as a guideline for single groynes. The ratio between length and spacing between the groynes is one of the most important factors of the effectiveness of a groyne field. In case groynes are spaced too far apart, the current may return to the bank before the following groyne in the system is able to influence the flow direction, resulting possibly in bank erosion or even loss of the next downstream groyne. If groynes are spaced too close, the groyne system would work less efficiently in controlling the flow and proper use of individual structure is not made. Moreover too closely spaced groynes are too costly.

General practice is to relate the spacing of groynes to their length. But the spacing also depends on the orientation to the flow velocity, the bank curvature, and purpose of the groynes. Usually the spacing is considered about 2 to 3 times the length of groynes. This rule includes some safety for the bank erosion. In favorable conditions and without additional safety a single groyne can protect 4 to 5 times the length of the groyne. An Indian guideline suggests a spacing of the groynes by 0.1 to 0.15 of the meander length of an outflanking channel. If the meander length is 15 to 30 times the channel width and length of the groyne is 30% of the channel width, then the spacing is 0.6 to 1.5 times the length of the groyne.

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

On concave outer banks, groynes are to be placed closer than on straight banks. The ratio should be reduced to around 2 to 2.5 on a convex bank. Often the length and spacing are finalized on the basis of the model tests.

In a general layout of a series of groynes all the groynes are similar regarding length, cross-section, orientation and the shape of the head of the groyne. However, often the most upstream groyne is attacked by the flow stronger than the other downstream groynes, and in that case the design of that groyne should be adjusted. In principle either the length of the groyne can be reduced, or the orientation can be changed or the top layers can be stronger than those of the other downstream groynes. Normally erosion of alluvial rivers take place along a long increasing bend; in that case a gradual increase of length may be necessary from the most u/s groyne to few following d/s groynes.

Often groynes are not only intended to protect the banks from being eroded but also, particularly in alluvial rivers, to increase the depth of the navigation channel by reducing low discharge channel width. In these cases, an optimal increase of the navigable depth is obtained if the spacing is two times the length of the groynes, because with that ratio just one eddy between two groynes gives a maximum guidance to the main flow of the river.

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

9.4 Layout Alignments

There is no general rule regarding the plan view of groynes. The choice is strongly influenced by the purpose which they have and whether used singly or in series. Groyne in series may be used if a long reach is to be protected and it also depends on their action of flow, deflecting or repelling.

Since there is no rule of general applicability for determining the plan view of groynes, it is advisable to have properly planned and performed model tests. This will mostly result in reliable information on shape and dimension of groynes.

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 up-stream embayment and to smooth out the transition between protected and unprotected reach downstream from the groyne field (Fig. 9.1). 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.

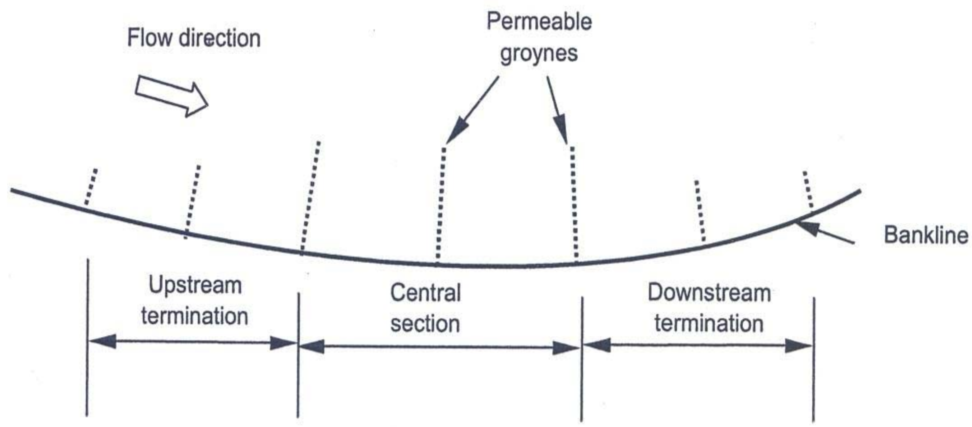


Fig. 9.1 Layout of Groyne Field

9.5 Cross Section Requirements

A typical cross-section of a groyne depends on many considerations. One of the most important factors is that whether the groyne should be conceived as permeable or impermeable.

a) Permeable Groyne

Permeable groynes are generally constructed of timber, steel or reinforced concrete piles driven or sunk into the river bed in one or several rows as shown in Fig. 9.2. The individual vertical piles are mainly subjected to horizontal loads, either on their total length by flow and wave attack or at changing heights due to floating debris, etc. In order to absorb the decisive horizontal loads (static and dynamic), it is advisable to have at least two rows of piles braced to each other by traverses and diagonals. The dimensions of the piles should be designed in accordance with the standard rules of structural engineering. The spacing between the piles should be at least one diameter; hence the minimum permeability would be 50% of the area. For a standard design, a maximum permeability of 80% is recommended, because of the effect on the flow field. Two parallel rows of piles can be placed in two different positions relative to each other. No recommendation is given for a certain arrangement.

Permeable groynes decrease near bank flow velocities creating rapid deposition in that area, particularly in alluvial rivers with considerable bed load and high sediment concentration. To prevent from flow separation and achieve a gradual deceleration of the flow velocities towards the river bank, the maximum permeability should be chosen 80% at the groyne head and decreasing up to 40% at the groyne root.

The advantages of permeable groynes are evident when applied in groyne field. A single groyne protects the bank against erosion only over a restricted area in the vicinity of the structure. Due to its sensitivity to changing directions of flow attack, a single groyne is not recommended for general application. Instead, groynes should be preferably used in series to protect a certain reach of the river bank. Permeable groyne fields have the advantage of reasonably controlling the flow pattern (similar to 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 flow velocity near the bank.

The driving depth of the piles depends mainly on the subsoil conditions, the local scour hole and the pile characteristics. A falling apron reduces the local scour between the piles and reduces the depth of piles driven into the soil. The height of the piles is normally 1 meter above the maximum design water level. In special cases the height of the piles can be reduced below this level which would result in a submerged permeable groyne during high floods.

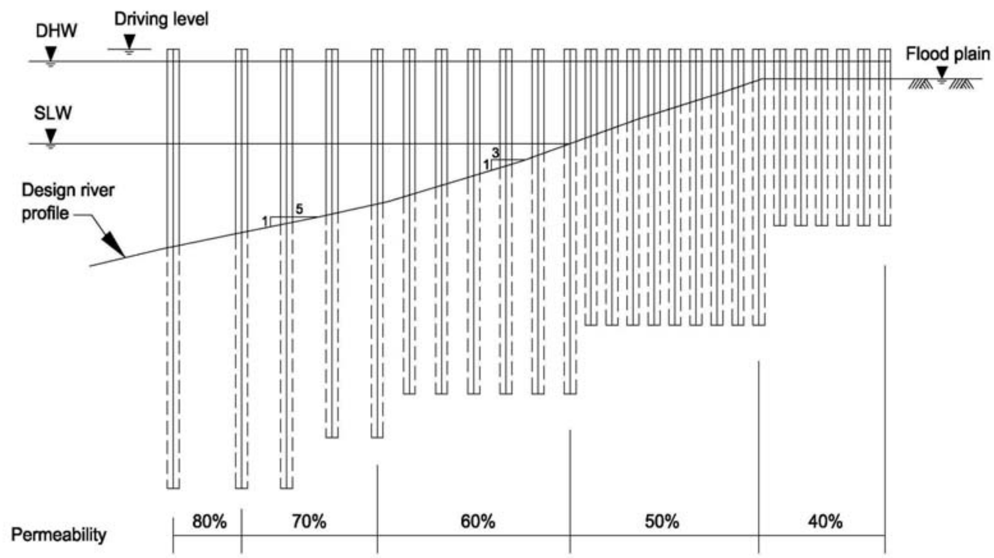


Fig. 9.2 Typical permeable groyne

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

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

b) *Impermeable Groyne*

Impermeable groynes can be built of local soil, stones, gravel and rock with suitable slopes at the shanks and the head or even vertical walls at the shanks. In case of an appropriately sloped earthen dam (Fig. 9.3), the trunk and the head have to be protected by a cover layer placed on a suitable filter layer.

Standard cross sections of the shank of an impermeable groyne have a trapezoidal shape: two side slopes and a horizontal top. The top of an impermeable groyne should have a minimum width of about 4.5m, so that a truck can pass for carrying construction materials. If the sides of the shank have a protective top layer, the slope should be designed as steep as the stability of the top layer. This allows minimizing the area of the top layer. A light falling apron or bed protection is recommended at the upstream side of the shank of the groyne.

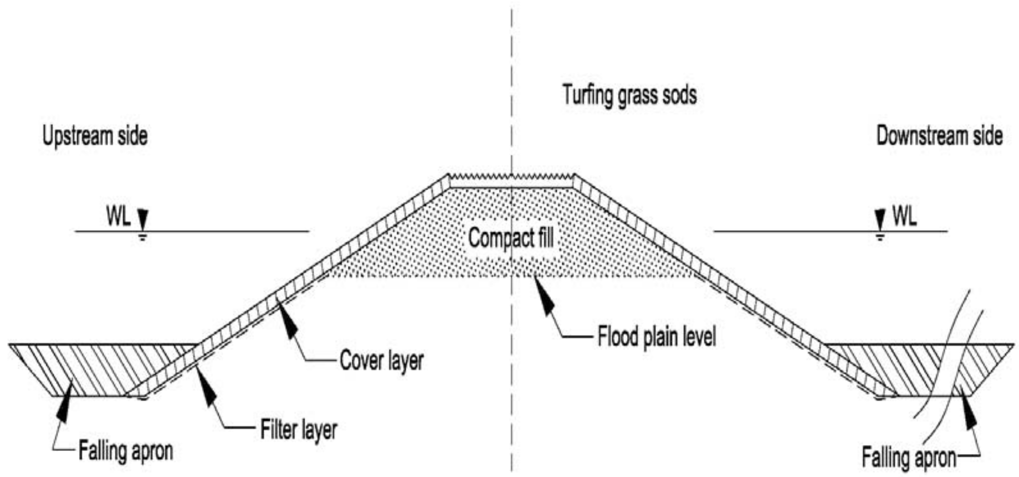


Fig. 9.3 Typical cross-section of an impermeable groyne

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

Vertical slopes can, for example, be made of steel or concrete piling as part of a cofferdam or of gabions. The space between vertical walls may be filled with locally available materials. The vertical walls can induce serious scouring of an alluvial river bed and require a massive bed protection around the head since the downstream of a groyne with a vertical head, strong eddies with high flow velocities are generated. For this reason vertical groynes are not recommended as a standard design.

Due to different construction methods above and below the low water level, the designs of the top layers of inclined slopes are often different above and below design low water level. The top layer below low water level must have a rather flat slope (1:3 to 1:4) for stability reasons or to reduce the scour depth, whereas the slope above design low water level may be steeper to save space and construction material. The transition from one top layer design to another is often less resistant to flow attack than the actual top layers. Therefore, a berm with a width of at least 1.5 m is designed at the transition.

The radius of the groyne head is determined by the effect of the head on the deflection of the flow, and the minimum radius may also be determined by the size of the top layer elements.

The crest of an impermeable groyne can be designed at standard high water level with a gentle slope of 1:100 towards the river along the axis of the groyne.

The existing river bed around a groyne should be protected from scouring by a bed protection or a falling apron. A bed protection has a length of about 1.5 times the scour depth at the downstream edge of the bed protection.

Impermeable groynes can be characterized as active structures, followed by extreme scour at the groyne head and severe erosion of the river bank by existing return currents. In a standard layout of a series of groynes, all the groynes should be similar with respect to length, orientation and shape of the groyne head as mentioned in Art. 9.3. However, often the most upstream groyne is attacked by stronger flow in comparison to the other groynes in the main section. In such case the design of the most upstream groyne should be adjusted by reducing the groyne length, changing the orientation or increasing the protection measures.

9.6 Design Considerations of R.C.C. Groynes

9.6.1 Introduction

The RCC groyne should have a special design method as it is not similar to the conventional groyne. The belmouth of conventional groyne is round and transition of flow is smooth. But the tip of RCC groyne is very sharp and create heavy turbulence when the flow hits the RCC wall.

The basic design criteria of I-shaped conventional groyne is followed for design of RCC groyne. In design of a T-head groynes the nose and straight portion has to be considered for calculation of scour depths at different locations or different part of the groyne. But in designing a RCC groyne, the groyne is considered as a cliff or wall protruded into the river. Accordingly the multiplying factor, $x = 2.25$ is considered to calculate the anticipated scour depth at the head of the RCC part. The design considerations are assumed on the basis of basic design of River Training works and morphological conditions of the rivers.

The used design methods including empirical equations, table, graphs, sample calculations usually practiced by the BWDB are described briefly in this chapter.

9.6.2 Design parameters

- Velocity
- Highest Flood Level and Lowest Water Level (HFL & LWL)
- Design Discharge
- Grain size of bed material (D_{50})
- Local scour, general scour and impact after construction (if model study is done)
- Present bathymetry
- Protrusion length of the groynes in to the river
- River bank line

The main decision is to take what type of River Training work would be useful to protect the bank erosion. RCC groynes were designed as a major bank protection works in the Ganges, the Jamuna, Teesta and Dudkumar Rivers in the last few years. After taking decision on the type of bank protection work the following decisions are to be taken:

Decisions before design

- Design has to compromise as per availability of fund. As per instances during design of Teesta left bank protection work, design had to change several times to reduce the cost as mentioned in the PP
- Whether model study (mathematical or physical) will be necessary for design
- Time of preliminary and final design
- What type of launching materials would be available or can be made suitable for construction?

Decisions during design

- Depth of anticipated scour depth
- Length of pile and volume of launching materials to meet the anticipated scour depth
- Size of launching materials and pitching blocks
- Thickness of cover layer of earthen shank
- Type and thickness of filter materials
- Design cross section of closure channel
- D/S slope protection
- Depth of sheet pile etc.

9.6.3 Length and spacing of the RCC groyne

Length: The length of the groyne depends upon the position of existing bank line and designed or expected bank line for trained river. Too long groynes on easily erodible rivers are susceptible to damage and failure (Garg, 2004). Experience of long groynes in Jamuna River also supports this. According to Varma et al (1989), the length of the groynes should be $L< 0.2B_{river}$. In construction of RCC groyne, the selection of length is a little bit different than defined in texts. To construct the RCC part, dry land (char land) is essential for casting situ-piles, pile cap, beams, wall etc. The near bank channel width dominates the length of the groynes. The length of the RCC groynes can be selected depending on the morphology and hydrology of the river condition.

Spacing: The spacing is taken as a certain proportion of their length. Apart from the length, the spacing is also governed by:

1. Type of bank where the groyne is located
2. The width of the river
3. Type of the groynes: permeable or impermeable groynes, shape and orientation of the groynes etc

Spacing can be selected from the specifications:

1. $S< 2.5B_{channel}$ (Varma et al, 1989), via spacing < 0.25 meander length = 2.5 width
2. $S<1$ to $2B_{channel}$ (Jansen et al, 1979)
3. Generalised expression for groyne spacing: $S<2.5K_1K_2K_3B_{river}$ (Varma et al, 1989)

Where:
K_1 groyne head
K_2 alignment curvature:
Straight $K_2 = 1$
Curved $K_2= 0.7$
K_3 function groynes
Bank protection $K_3 = 1$
Navigation $K_3 = 0.5$

In designing of RCC groyne, spacing of 2 to 2.5 times of the length of groynes are generally provided, depending upon the characteristics of river morphology and model study.

9.6.4 Bed Protection

Groynes designed for bank protection may develop a scour hole at its head due to disturbance in the flow. Moreover, heavy scour may occur downstream due to eddy formation. Therefore, groynes should have a strong head to resist the direct attack, but also the river bed at groyne heads and flanks are to be protected by a sufficiently designed apron, the thickness and dimensions of which can be reduced from the head to the bank since the scour diminishes accordingly.

The scour holes at the head of a permeable groyne are not as serious as with the impermeable groynes and this event can be influenced by a favorable distance of piles. The material of apron such as stones, boulders, sand filled geo-bags etc must be sufficient to shield the slope of the scour hole down to the deepest level.

The detail design of bed protection is provided in chapter 8 under falling apron.

9.6.5 Typical design of a groyne in Ganges River

(a) Design Parameter

Following are the major design parameters used for the design of the groynes

1. Max Discharge = 71600 m^3/s (max discharge in 1998, According to FAP25: 1 in 25 years discharge is 76,000 m^3/s, IWM report, June 2002)
2. Design Discharge = 71600*0.7 = 50120 m^3/s (Dominant discharge)
3. Grain size of bed material = 0.12 mm
4. Silt factor, $f = 1.76\sqrt{D_{50}} = 1.76 * \sqrt{0.12} = 0.606$
5. HWL = 23.23 m (PWD)
6. LWL = 13.44 m (PWD

(b) Calculation of scour depth:

Scour depth by Lacey's Eq., $R = 0.47 * \left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47 * \left(\frac{50120}{0.606}\right)^{\frac{1}{3}} = 20.47m$

Considering the multiplying factor, X=1.75 (for severe bend)
Then anticipated scour depth, R*X= 20.47*1.75 = 36m
Therefore, anticipated scour level = (23.23 – 36.00) = (-) 12.77m (PWD)
Observed average bed level = + 12.58 m

(c) Calculation of revetment thickness:

1. Inglis Eq. $t = 0.06 * \left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.06 * \left(\frac{50120}{0.606}\right)^{\frac{1}{3}} = 2.94m$
2. t = 28" = 0.71m (For medium size bed material, by Spring)
3. t = 3' 6" = 1.067 m (For 1.5 to 2.5 million cusec = 42,500 to 71,000 m^3/s discharge by
Gales)

Let, t = 1.067 m for our case

(d) Size of hard material:

From graph for velocity, V = 2.5 m/s, C. C. Block size = 30 cm diameter (Velocity from IWM model study)

Computation for Block Size:

Considering Stability against Current attack Using Neill's and JMBA Equations

Data:

Maximum near bank velocity, V	: 3.00 m/sec
Specific gravity of stone, S_g	: 2.60
Depth factor, h/d	: 5
Slope angle of bank, θ	: 26.56° (1V:2H)
Angle of repose of revetment material	: 40º

The depth average velocity has been estimated 2.00 m/sec as derived from hydrodynamic model results corresponding to 1988 flood flow. Considering factor of safety, depth average velocity 3.00 m/sec has been assumed as design velocity for the recommended interventions.

Computation:

(i) Neill's Equation, D = 0.034 v^2 = 0.034 $*(3.00)^2$

= 0.39 m (dia of stone) or 0.310 m (size of cubical block)

(ii) JMBA equation

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

$$D_n = \frac{0.7(3.00)^2}{2\cdot(2.60-1)\cdot 9.81}\cdot\frac{2}{\log(6*5)^2}\cdot\frac{1}{\left[1-(\sin 26.56/\sin 40)^2\right]^{0.5}}$$

= 0.36 m (size of cubical block and corresponding weight is 112 kg)

Use 40cm*40cm*20cm size block for slope protection above LWL and 450 mm (40 %) and 400mm (60%) size cubical block below LWL and toe protection. The corresponding weight of the pitching block is 76.8 kg which is less than 112 kg.

(e) Calculation of Pile length:

Max discharge = (71600 +283) m^3/s (283 m^3/s discharge from Boral)

= 71,939 m^3/s

Design Discharge, Q = 0.7*71939 = 50357 m^3/s

Scour depth, $R = 0.47*\left(\frac{Q}{f}\right)^{\frac{1}{3}} = 0.47*\left(\frac{50357}{0.606}\right)^{\frac{1}{3}} = 20.51m$

Then anticipated scour depth, R*X= 20.51*1.75 = 35.89 m from HWL

Therefore, anticipated scour level = (23.23 – 35.89) = (-) 12.66 m (PWD)

Provided Pile bottom level = (-) 14 m (PWD) and the corresponding Pile length 24 m

[**Comments**: The multiplying factor should be minimum 2.25 or more at the head of RCC groyne. The groyne head is like a cliff or wall in to the river. As per BWDB design manual: Table 9.2, factor 1.75 should be used for severe bend in case of revetment work]

(f) Steel sheet pile:

3.75m length steel sheet pile provided from the bottom of the deck slab to protect seepage for minor head difference.

(g) Volume of launching material at the head of RCC part:

Thickness of slope pitching, t = 1.067 m
Considering the slope of launching = 1: 2
Depth of scour, D = anticipated scour depth from the river bed = R*X–(HWL-LWL)

Therefore, volume of launching materials required, $V = \sqrt{5} *1.25*t*D = 2.8*t*D$

Table 9.1 Volume of launching materials at Panka Narayanpur groynes (after BWDB design)

Groyne No	Calculated scour depth, D (m)	Launching vol. 2.8*t*D (m^3/m)	Volume Calculated (m^3/m)	Volume provided (m^3/m)	Remarks
1	25.71	2.8*t*D	77	90	Grading of material 50%-CC block 45 cm cube 50% 30 to 45 cm diameter hard rock
2	22.44	2.8*t*D	67	90	
3	31.62	2.8*t*D	95	100	
4	23.76	2.8*t*D	86	100	
5	37.00	2.8*t*D	110	105	
6	27.83	2.8*t*D	83	90	
7	27.71	2.8*t*D	83.0	90	
8	35.72	2.8*t*D	106.0	105	

(h) Computation of Local Scour:

1. Ahmed formula: Formula for local scour, $d_s = 4 * Y_0 * F_r^{0.33}$

Where, F_r = Froude number, Y_o = Avg. u/s flow depth

Table 9.2 Value of local scour at Panka Narayanpur groynes (after Ahmad, 1953)

Groyne No	Average U/S flow depth, m	Mean flow velocity, m/s	Froud Number $F_r = U_0 / \sqrt{gh}$	Scour depth (m) (by formula)	Remarks
1	9.37	1.55	0.16	20.56	
2	9.59	1.07	0.11	18.51	
3	9.36	0.86	0.09	16.93	
4	7.28	1.51	0.18	16.50	
5	10.40	1.82	0.18	23.63	
6	9.33	1.36	0.14	19.60	
7	7.10	1.42	0.17	15.83	
8	7.80	1.63	0.19	17.92	

Design of scour level:

Computation for Local Scour and Design Scour Level Using Formula of Ahmed (1953):

Data:

Water Level	: 23.30 m PWD
Minimum bed level	: 8.42 m PWD
General scour depth	: 4.32 m
h_1 = water depth	: 14.88 m
u_1 = u/s depth averaged velocity	: 2.18 m/sec
K = 2.60	

Computation:

Local Scour: $h1 + y_{s,local} = K\{h_1 U_1 * B_{ch} /(B_{ch}-b)\}^{2/3}$

$= 2.60\{14.88*2.18* 1.2)\}^{2/3}$

$= 2.60(38.93)^{2/3}$

$= 29.86$ m

$y_{s,local} = 29.86- h_1 = 29.86- 14.88 = 14.98$ m

Design scour depth:

=Local scour depth + General scour depth

=14.98 + 4.32 = 19.30 m

Design scour level:

=Bed level-Design scour depth

= 8.42-19.30 = -10.88 m (PWD), Use (–)12.00m (PWD)

2. Liu et al approach:

Local scour, $y_{s,local} = K_l h_0 \left(\frac{b}{h_0}\right)^{0.4} * F_r^{\ 0.33}$

Where, F_r = Froude number, h_o = Avg. u/s flow depth, b= protrusion length into the river, K_l= 1.1 co-efficient for streamline abutment (Comments: The tip of RCC groyne is a round shape like an I-shaped conventional groyne. So the flow line can’t be streamline like an I-shaped groyne).

Table 9.3 Value of scour depth at Panka Narayanpur groynes (after Liu et al)

Groyne No	Average U/S flow depth, m	Mean flow velocity, m/s	Froude Number $F_r = U_0 / \sqrt{gh}$	Protrusion (m) Length, b (m)	Scour depth (m) (by formula)	Remarks
1	9.37	1.55	0.16	467	26.84	
2	9.59	1.07	0.11	498	24.53	
3	9.36	0.86	0.09	200	15.71	
4	7.28	1.51	0.18	200	16.97	
5	10.40	1.82	0.18	200	21.08	
6	9.33	1.36	0.14	301	21.49	
7	7.10	1.42	0.17	344	20.44	
8	7.80	1.63	0.19	395	23.55	

Part – III

Construction

CHAPTER TEN

Earth Works

10.1 General Description

Earthworks are important elements of almost all civil engineering works including river training and bank protection works. Most of the earth works in river training and bank protection works have

- "excavation" as defined under Section 10.2.2, and
- "filling works" as defined under Section 10.2.3

10.1.1 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. Subject to availability of construction earth nearby or facilities for disposal of spoil within a head load area, the estimate can be framed solely on labour force for cutting, carrying, placing in place and also disposal of spoils. For any variation either in availability of soil for construction near the construction site or facility for disposal of spoil within head load distance, mixed means through engaging labour force and equipment may be employed.

Mechanical excavation, transportation and filling can accelerate the work in progress, but the decision should be made by considering 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. Table 10.1, 10.2 and 10.3 provide rough guidelines and ideas to estimate the productivity of mechanical equipment for excavation works. It has to be recognized that the actual productivity depends on several important factors and parameters which includes boom length, turning radius and angle of the excavator, equipment availability, maintenance etc.

Table 10.1 Excavation capacity in m^3/hr for different buckets (V = net bucket volume, top flat)

Consistency Classification of soil	Dragline Bucket			Clamshell Bucket		
	$V=0.5m^3$	$V=0.9m^3$	$V=1.5m^3$	$V=0.5\ m^3$	$V=0.9m^3$	$V=1.5m^3$
soft / loose	32	52	80	25	40	65
Medium stiff / medium dense	20	34	52	15	25	40
Stiff / dense	14	24	38	10	18	35
Underwater	About 60% of above values			About 60% of above values		

10.1.2 Wet Excavation

"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 10.1, but due consideration must be given to the reduction of hourly productivity to about only 60% as compared to dry excavation with the same equipment.

Table 10.2 Bulldozer transport capacity (m^3/hour); (D = Transfer distance)

Bulldozer capacity (HP)	Shield width (m)	Medium stiff / medium dense			Stiff / dense		
		D=20 m	D=60 m	D=100m	D=20 m	D=60 m	D=100m
45	2.20	80	30	20	unsuitable		
100	3.35	165	75	50	70	35	25
150	3.90	250	110	80	100	50	35

Table 10.3 Consistency classification

Soil type	**Consistency Classification**	**Description**
Cohesive	Very soft	Thumb penetrates easily; extrudes between fingers when squeezed
	Soft	Thumb will penetrate through soil 25 mm or more with ease; molds with light finger pressure
	Medium stiff	Thumb will penetrate about 6 mm with moderate effort; molds with strong finger pressure
	Stiff	Thumb indents easily and will penetrate 12 mm with great effort
	Hard	Thumb will not indent soil, but thumbnail readily indents it
Cohesionless	Very loose	Easy to penetrate with a 12 mm diameter rod pushed by hand
	Loose	Difficult to penetrate with a 12 mm diameter rod pushed by hand
	Medium dense	Easy to penetrate 300 mm with a 12 mm diameter rod driven with a 2.3 kg (5 lb) hammer
	Dense	Difficult to penetrate 300 mm with a 12 mm diameter rod driven with a 2.3 kg (5 lb) hammer
	Very dense	Penetrates only about 150 mm with a 12 mm diameter rod driven with a 2.3 kg (5 lb) hammer

10.1.3 Compaction Equipment

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

It is difficult to plan and decide on the compaction method and equipment for bank protection works because of the usual large variation in the soil available in the immediate vicinity of a construction site, including adjacent chars. Ordinary compaction methods can be considered appropriate under the circumstances and in consideration of what is commonly available from equipment and technical 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 10.4 may be used as a guideline only for determining the equipment resources and capacity.

Table 10.4 Capacity of various compaction equipments (typical for soil usually encountered along the rivers of Bangladesh)

Method of compaction/ Type of equipment	Suitable for compaction of	Weight (ton)	Layer thickness (cm)	Number of passes	Compaction volume (m^3/hour)
Tamping plate/ crawler crane	Non-cohesive soil	2 to 3	50 to 70	3 to 4	25 to 50
Heavy duty internal combustion tamper	Cohesive/ non-cohesive soil	0.5	30 to 40	3 to 4	15 to 20
		1.0	40 to 50	3 to 4	25 to 30
Surface (plate) vibrator	Non-to slightly cohesive soil	0.5 to 0.7	40 to 50	3 to 4	24 to 30
		> 1.7	50 to 70	3 to 4	40 to 50
Vibro-roller (self driven)	Non-to slightly cohesive soil	1.2 to 2.3	30	4 to 6	35 to 50
		3 to 6	40 to 50	4 to 5	90 to 110
Pneumatic roller (self driven)	Cohesive/ non-cohesive soil	9 to 12	15 to 20	8 to 12	65 to 100
Tamper roller (sheep foot roller)	Cohesive soil, free of stones	3.5	18 to 20	8 to 12	32 to 48
		6	18 to 20	8 to 12	65 to 95

10.2 Sample Specifications

Earthwork materials are defined to apply to the specifications and the works as follows:

- “top soil” is the top layer of soil that usually contain vegetation and other organic material;
- “suitable material” comprises all material obtained from excavations within the site or from borrow pits and which is approved by the engineer as acceptable for use in the works;
- “unsuitable material” is any other than suitable material and shall comprise:
  - (i) material from swamps, marshes and bogs or other material with a high organic content;
  - (ii) peat, logs sumps and perishable materials
  - (iii) material susceptible to spontaneous combustion;
  - (iv) 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.

10.2.1 Clearing of Construction Site

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

Notwithstanding the provisions of this subsection, the grass cover on existing embankments, that fall within the site and which have been designated on the Drawings or directed by the engineer for retention in their present state, shall be preserved.

10.2.2 Excavation

Definition

Excavation shall mean the removal of any type of material for construction / borrow pit, bank slope construction, bed preparation of revetment structures etc., whether in dry or in wet condition, so that the structures can be constructed to the lines, grades and dimensions shown on the drawings or as instructed by the engineer / designer.

Execution of the work

At all times during construction, the excavation shall proceed in a way which ensures that the stability of any slope will not be impaired. The excavation procedures and stability of all slopes excavated shall be maintained in a way so that throughout the construction period a sound construction environment is maintained. The engineer's / designer's approval of excavation procedures shall in no way relieve the Contractor of his responsibility for safeguarding the stability of all slopes excavated under the contract.

The excavation shall be carried out in such 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. Only those materials meeting the appropriate specifications may be incorporated into the permanent works. Unsuitable material shall be removed from the site.

With the exception of reasons falling under force majeure all excavations meant to be kept dry shall be maintained free from water arising from any cause. Any surface or subsurface water flow, entering into or adjacent to the excavation, shall be controlled as desired by the designer / engineer.

Where required, the sides of excavations shall be maintained with proper support.

Tolerances

Excavated levels shall be within a tolerance of ±100mm. Slopes of embankments and cuttings shall be accurate within ±100mm to 150mm within 10m measuring distance.

If for any reason whatsoever excavations were carried out beyond permissible tolerances to their true lines and levels as shown on the drawings or as desired by the designer / engineer, the excavation shall then be restored to the required lines and levels with compacted sand or other approved material, and in such a manner as the engineer may direct.

Approval of Excavation

After any excavation is completed accurately to the lines and levels required for the works, the engineer / designer shall verify and check such excavation.

The subsequent work shall commence only after the approval of the excavation of depths and slopes, including the compaction of the excavated surfaces, where required, by the engineer / designer.

The open excavations shall be maintained in an approved condition. Where required, the effects of deterioration due to surface water or any other act shall be rectified.

10.2.3 Filling Works

Definition

Filling works comprise of filling and compaction of any land, construction of embankments or groynes, filling and compacting of construction pits and excavations and back filling of structures to designed levels.

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 non-plastic fine (material passing #200 US sieve, grain size less than 0.074 mm) content of 5% for 'well graded sand' (i.e. $C_u \geq 6$ and $1 \leq C_c \leq 3$) and 10%for 'poorly graded sand' (i.e. sand not meeting the C_u and C_c values for well graded sand). Here, uniformity coefficient, $C_u = d_{60} / d_{10}$ and Coefficient of curvature, $C_c = d_{30}^2 /(d_{60} x d_{10})$.

Suitable material for construction of embankments shall be obtained from excavation from designated borrow pits adjacent to the site or dredging.

The unsuitable material covering the borrow pits shall be removed to a depth where suitable material can be obtained. Unsuitable material shall be temporarily stored and refilled to the borrow pits after completion of the works, or otherwise utilized as directed by the engineer / designer.

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 unsuitable material, all as per specification and requirements in consideration to stability.

Representative samples of the fill material for testing shall be taken during placement. 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 will be rejected and shall be removed from Site. 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.

Compaction

The type of compaction equipment employed shall be such that the required densities can be obtained consistently.

Trial compactions for each type of fill material shall be performed in order to establish the appropriate method to achieve desired compaction level. Based on the results of such trial compactions, the necessary number of passes of compacting equipment and layer thickness shall be determined in consideration of the type and weight of compaction equipments used.

Before starting any filling works, the state of density (in terms of Relative Compaction, RC) of the natural ground has to be determined by the Contractor to the Employer's requirements. The engineer / designer will thereupon decide on the necessity of compaction of the existing soil.

$$\textit{Relative Compaction (RC)} = \frac{(\gamma_d)_{field}}{(\gamma_d)_{\max}} \mathrm{X}\, 100$$

$(\gamma_d)_{field}$ = Dry density in the field (ASTM D1556 – test method for density of soil in place by the sand-cone method or any other standard method)

$(\gamma_d)_{\max}$ = Maximum dry density by laboratory test
(For clay soil - ASTM D1140, Test method for laboratory compaction characteristics of soil using standard effort; For sand - ASTM D4253, Test method for maximum index density and unit weight of soils using a vibratory table)

The required density values of the fill 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 compaction equipment and results of compaction tests, taking into account an appropriate safety margin.

The achieved state of compaction of each layer of fill shall be verified as the filling and compaction work proceeds and for this purpose Relative Compaction, RC of each layer of fill shall be checked. For each 500 m^2 of compacted surface three spots, selected at random or from areas of apparently reduced compaction, are to be investigated by density tests.

The results of all such tests shall be recorded and preserved in approved form.

The required compaction level is as follows:

Layer	Relative Compaction RC (%)
Up to 0.5m below present surface	≥ 90
Fill for embankments, dams	≥ 95

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, re-compaction of such area shall be carried out and new tests executed thereafter.

Portions of fill which 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.

CHAPTER ELEVEN

Filters

11.1 General

Filter is an essential integrated part of all types of protection works. Protective filters placed on less pervious subgrade soil on a slope facilitate the flow of seepage water and prevent erosion of soil and thus damage to overlying structures (rip-rap, revetment etc) from uplift pressure. There may be mainly two types of filters i.e. 'granular filters' (section 11.2) and 'geotextile filters' (section 11.3).

11.2 Granular Filter

11.2.1 General Description

Granular filters can be either 'zoned filters' or 'mixed filters' suitably graded to contain the base material. The former, consists of at least one sand layer and one gravel layer (each layer with an appropriate gradation) and can only be placed above water level. Immediately after trimming of the embankment slopes, placement of the filter layer shall start at its toe in horizontal layers and in such a manner that washing out and segregation of different sizes of particles is prevented. Prior to laying the subsequent layer, the achieved slope and layer thickness of the initial layer must be confirmed through measurements.

The 'mixed granular filter' is a suitably graded one-layer filter with wide range of particle size. 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. Placement of mixed granular filter can be accomplished by one of the following methods:

- using large diameter tremie-pipes to release the graded material directly at the bank slope or river bottom;
- using large size buckets, which can be lowered by crane and opened directly above the river bed and by gently swinging the crane's 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 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.

Another relatively new application of mixed granular filter are geo-bags filled with sand or mixed filter material dumped at a controlled pattern to achieve at least 2 layers of bag coverage. Properly designed size of bags filled with graded material (sand and gravel/ khoa) can serve as filter as well as apron material.

11.2.2 Materials for Granular Filter

Zoned filter comprise of at least two layers:

- Sand layer : Fine filter material (sand);
- Gravel layer: Coarse filter material (gravel or stone chips or khoa).

For application below water level a one-layer granular filter (mixed filter) may be applied, the grain size range and gradation of which must be equivalent to the corresponding two layer system in terms of performance.

11.2.3 Fine Filter Material (sand):

Fine filter shall comprise of sand and comply with the grading shown in the drawing. 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.

11.2.4 Coarse Filter Material (gravel, stone chips or khoa):

Coarse filter material shall be made from either:

(a) *Gravel (shingle) or broken stone chips*, the stone chips or gravels shall be granular and free from all organic impurities. The grain size distribution may generally be in the range of D_{15} = 6.0 mm to D_{60} = 100 mm, but must be suitably adjusted to grading of sand filter and cushion layer.

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 material. Any dust or fine material below 5 mm in size made in the stone crusher is to be removed by screening.

(b) *Khoa (crushed brick*), the khoa filter material shall be prepared from first class bricks or picked jhama chips only. The grain size components must be carefully produced, selected and blended to suit the required grading.

11.2.5 Mixed Granular Filter:

Mixed granular filter shall mainly be granular, cohesionless, and free from organic impurities. The grain size distribution will generally be in the range of D_{15} = 0.4 mm to D_{60} = 100 mm, but must be suitably adjusted to the grading of the subsoil to be protected.

11.2.6 Foundation Preparation and Laying Filter:

The foundation for the filter materials shall be thoroughly compacted and graded to the elevations shown on the Drawings prior to the placement. The filter material shall be placed in a uniform thickness in each layer as shown in the drawing and each layer shall be lightly compacted before placing the second layer. Placing or constructing filter shall start from the toe of the embankment or lowest line shown in the design drawing and progress upwards. A maximum of 1.0 m of laid filter works shall be covered with the protection element before progressing further upwards.

The placement of mixed filter shall be carried out as per standard procedures or procedure approved by the engineer in charge.

11.3 Geotextile Filter

11.3.1 General

Geotextile fabric used for the filter layer below the slope protection shall be a non-woven geotextile of the staple or continuous fiber type or similar material approved by the Engineer.

Geotextile, whether geotextile sheets or tailored geotextile fabrics shall be made of a reputable manufacturer, and the manufacturer has to produce test certificates of a recognized testing institute to confirm compliance with the specified requirements.

Before placing an order for any quantity of geotextile, test reports for each type of geotextile from an independent testing laboratory must be obtained.

Brand name and grade of all geotextiles shall be clearly and uniformly marked on the upper face. The marking shall take the form of an indelible repeat roll imprint at the edge of each geotextile roll recurring at least every 1.5 m.

Geotextile shall be placed on the slope of an embankment or on bank slope in strict compliance with the manufacturer's instruction and approval of the Engineer. This holds in particular for the method of jointing, overlapping and laying out.

When laying geotextile sheets, the surface of the embankment /bank slope should be relatively smooth plane, free from obstructions, depressions and soft pockets of materials. Depressions above water must be filled with compacted material. Filling of underwater depressions should be avoided as much as possible. In unavoidable circumstances cutting or leveling protrusions to prepare a smooth plane is the desired solution. Another acceptable way is to fill underwater depressions on embankment or bank slopes with granular material filled in jute / synthetic or geo bag.

The cover layer shall be placed as soon as possible. The material of the cover layer shall be deposited starting from the bottom of the slope.

11.3.2 Geotextile used as Revetment Filter

Protective layers of uniformly sized concrete blocks or stone mattresses require filter to perform the function of filtration, drainage, separation and protection. A fabric filter (non-woven needle punched geotextile) of the following specification may serve the minimum requirement for most of the soil type encountered along the major rivers of Bangladesh.

Table 11. 1: Geotextile properties as revetment filter

Material: Preferably 100% polyester (PES) / 50% polypropylene (PP) and 50% PES	
Characteristic opening size O_{90} (ISO 12956)	≤ 0.08 mm
Thickness (under 2 kPa pressure) (ISO 9863, ASTM D1777)	≥ 3.00 mm
Mass per unit area (ISO 9864, ASTM D3776)	≥ 350 gm/m^2
Wide width tensile strength (ISO 10319, ASTM D4595)	≥ 20 kN/m
Grab strength (ASTM D4632)	≥ 1200 N

CBR puncture resistance (ISO 12236, DIN 54307)	≥ 3500 N
Elongation (both direction) (ISO 10319)	≥ 40% and ≤ 90%
Permeability (both direction, under 2 kPa pressure) (ASTM D5396, ASTM D4716)	≥ $2x10^{-3}$ m/sec

11.3.3 Geotextile used as Sub-layer for the Launching Apron

For the use of geotextile underneath a launching apron (articulating mattress) the material must be able to resist larger tensile stresses. The fabric (non-woven needle punched) should have the following specification:

Table 11.2: Geotextile properties as revetment filter

Material: Preferably 100% polyester (PES) / 50% polypropylene (PP) and 50% PES	
• Characteristic opening size O_{90}	≤ 0.08 mm
• Thickness (under 2 kPa pressure)	≥ 4.00 mm
• Mass per unit area	≥ 500 gm/m^2
• Wide width tensile strength	≥ 25 kN/m
• Grab strength	≥ 1600 N
• CBR puncture resistance	≥ 4500 N
• Elongation (both direction)	≥ 40% and ≤ 90%
• Permeability (both direction, under 2 kPa pressure)	≥ $2x10^{-3}$ m/sec

11.3.4 Transport, Storage, Handling and Placing of Geotextile

(1) All geotextiles shall be transported, handled and stored in full accordance with the manufacturer's instructions. They shall be wrapped in black polyethylene sheeting to prevent UV exposure until immediately before use in the Works. If the wrapping is damaged during handling it shall be repaired immediately using additional black polyethylene sheeting. Unused portions shall be re-wrapped promptly.

(2) Geotextile fabrics arriving on site in containers shall be unpacked and stored under covers, well sheltered from rain and 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.

(3) Torn or punctured geotextile fabric shall not be permitted in the Permanent Works.

(4) Joining of geotextile shall be machine seamed (prayer seam) joints with specified polypropylene or nylon thread. Overlaps on the slope shall be arranged down the slope. The upstream edge of the geotextile shall be always below the other one. If an overlap transverse to the slope is unavoidable, then the upper geotextile should be laid on the top of the lower one. Overlaps shall be at least 40 cm when placed in the dry. When placed under water the overlapping should be minimum 1.0 m for water depth up to 4.0 m and minimum 1.5 m for water depth up to 8.0 m.

(5) Overlaps (underwater) are to be controlled by divers immediately prior to placing the cover layer.

(6) Geotextiles are to be covered with suitable materials 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 2m.

(7) During installation of geotextiles and subsequent placing of cover layers no high stresses shall be exerted on the geotextile and at no situation shall a geotextile be pulled around or along sharp edges. The installation equipment must be suitably designed to fulfill this requirement.

(8) When geotextiles are installed on a slope, the cover material shall be deposited starting from the toe of the slope.

(9) Stock piles of materials are not to be set on top of laid geotextiles unless the geotextile has been designed for such loads.

(10) No construction equipment is to work on the geotextiles without at least 300 mm of suitable material overlying the geotextile.

11.3.5 Geotextile Testing

(1) In general testing of geotextiles will follow ISO standards in their latest version The geotextiles to be incorporated within the Works shall comply with the appropriate codes and standards including the following:

ASTM D4491:	Standard test methods for water permeability of geotextile by permittivity.
DIN 53936(pt1):	Determination of the water permeability coefficient k_{v1} normal to the geotextile plane with constant head
IS0 9073-1:	Determination of mass per unit area for non woven textiles.
IS0 9073-2:	Determination of thickness of non-woven textile
IS0 9073-3:	Determination of tensile strength and elongation of non-woven textiles.

(2) The filter characteristic opening size, 0_{90}, defined as being the grain size of a standard sand corresponding to 90% retention by weight on a sample of the geotextile in a vibrating sieve apparatus, shall be measured in a wet apparatus using the BAW (Bundesanstalt fur Wasserbau - German Federal Institute for Waterways Engineering) method.

(3) Testing of abrasion resistance shall follow the standard rotating drum test of Bundesantalt fur Wasserbau (Federal Waterway Engineering and Research Institute, Germany). The remaining strength after the abrasion test is defined in Guidelines for Testing Geotextiles for Navigable Waterways (RPG) 1994.

(4) Tests of mass per unit area, thickness, and tensile strength in accordance with the Standards listed in Clause 11.5(1) shall be carried out by an approved testing laboratory

on samples taken from each quantity of 10,000 m^2 of geotextile fabric supplied. The values of k and O_{90} values shall be tested on samples taken from every 50,000 m^2 of geotextile fabric supplied. Seams shall be tested for tensile strength every 10,000 m of seam.

(5) The geotechnical test results of the underlying embankment soil together with the manufacturer's specification and installation instructions for the proposed cloth, including permeability and porosity (with methods of testing) and a sample of the cloth shall be submitted for the approval of the Engineer.

(6) The sample size for the fabric shall be 2 m^2 and shall be marked to indicate its upper side, longitudinal and transverse directions, type of geotextile and the date that the sample was taken. Seam samples shall be at least one meter in length and the ends of the threads are to be firmly tied off at the time the samples are taken. Each test shall be carried out on at least five samples.

(7) Notwithstanding the submission of reports to the effect that the geotextile conforms to the Specification, the Engineer shall at all times be entitled to have additional samples of geotextile tested if he is of the opinion that the geotextile does not conform to the Specification. The Engineer shall only select samples from ends of geotextile rolls or geotextile which has been cut already.

(8) A geotextile will be regarded as defective if any of the specified values are not achieved other than those of unit weight and effective opening size, for which the following tolerances will be permitted:

(a)	single layered geotextiles:	
	unit weight	- minus 10%
	0_{90}	- plus or minus 20%
(b)	composite geotextiles:	
	total weight	- minus 15%
	single layer weight	- minus 20%
	0_{90}	- plus or minus 20%

CHAPTER TWELVE

Rip-Rap and Bonded Stones

12.1 Rip-rap

One of the most common types of protective cover layer is rip-rap, which generally comprises well graded and randomly placed stone or rock. The brick blocks or cc blocks placed as cover layer for protection work are also a form of rip-rap. In addition to the mentioned cover layer protection Wiremesh Mattressing and Grout Filled Mattress are also used for the purpose of protection. It has the following advantages:

- easy 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 normal size and the grading range of the material depend on the hydraulic loads and can be determined by application of the relevant formulas for current attack and wave attack. The best suited are angular, nearly cubical stones. The ratio of the maximum to the minimum dimension of a stone should not exceed 2.5.

The specific gravity of the stones can vary from 2.0 ton/m^3 for sedimentary rocks up to 2.9 ton/m^3 for igneous rocks, but is typically in the range 2.5 to 2.7 ton/m^3.

The mixture of stones should form a smooth grading curve and for optimization of the stability of the cover layer it is recommended to keep the gradation within the limits shown in the respective grading diagrams (Figure 12.1), 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 dumping the riprap on the top 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 localized stresses or to provide a granular sublayer of smaller stones or gravel to act as a protective sub-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 depth and velocities and requires relevant experience of the team conducting the work.

To enhance the stability of rip-rap against hydraulic forces, bonding by colloidal cement grout of sand-bitumen mix is 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.

12.2 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 three basic methods of grouting (Figure 12.2):

- 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 weep-holes 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%.

12.2.1 Cement-Grouted Rip-Rap

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 colloidal grout.

Table 12.1 Grading Range C (source: WARPO 2001) (suited for rip-rap layers with bitumen pattern grouting)

Range C	Stone 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 12.2 Grading Range D (source WARPO 2001) (suited for rip-rap layers with surface grouting by colloidal cement grout grouting)

Range D	Stone 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 12.3 Grading Range E (WARPO 2001) (suited for rip-rap layers without additional bonding)

Range E	Stone 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

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}

The bonding is to be carried out as surface grouting, whereby, an adequate permeability of the cover layer has to be guaranteed by controlling the quantity of grout placed per unit area. The cement grout has to be poured in 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} = 25cm, Grading Range D, layer thickness ≥ 50cm) above water level, where the stone material can be placed well and relatively dense, at least 100 liters grout per square meter of surface area are to be applied.

For above-water areas, the cement grout should be applied 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 structures with loose stone cover layers shall be grouted with 50 liters/m^2 for a width of 10m.

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. 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 composition:

Table 12.4 Grading of aggregates for cement grouted rip-rap layers

Grain size [mm]	Percentage passing the test sieve (by weight)
0.25	≤ 25
0.50	≤ 60
2.00	≥ 90
4.00	100

The water used for cement grout mixing shall be 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.

Prior to starting the work trial mixes shall be carried out. 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 20x20x20cm, and a comprehensive test shall be performed. 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 20x20x20cm after 28 days of pouring.

- Characteristic strength (minimum compressive strength of each cube): 25 N/mm^2
- Series strength (minimum compressive strength of each series of 3 cubes): 30 N/mm^2

For each day on which grouting works are being carried out at least three test specimens (size 20x20x20cm) 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:

Diameter of grout spread on the table (mean value as measured in two directions):

One minute after lifting the mould, without striking the flow tables: $a = 32 \pm 2$cm

When measured immediately after striking the flow table 15 times: $a = 48 \pm 2$cm

12.2.2 Bitumen-Grouted Rip-Rap

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.

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 litre/m^2
- layer thickness 50 cm : 120 to 140 litre/m^2

Trials are to be carried out at site to determine the practical quantities of grout required in accordance with the realistic volume of voids of the placed stone cover layers.

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.

Certificate of origin of the bitumen as well as respective factory test certificates or bitumen samples tested at a recognized material testing institute shall be produced before commencement of work, to prove compliance with the Standard.

Clean sand of grading range 0-2 mm is to be used as aggregate.

Table 12.5 Grading of aggregates for bitumen-grouted rip-rap layers

Grain size [mm]	Percentage passing the test sieve (by weight)
0.25	≤ 25
0.50	≤ 60
2.00	≥ 90
4.00	100

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:

Table 12.6 Grading of filler for bitumen-grouted rip-rap layers

Sieve size BS 410	Percentage passing
600 micron	100
180 micron	100-95
75 micron	100-80

The composition of bituminous grout shall be in the following range:

Table 12.7 Composition of bituminous grout

Component	Composition percentage by weight	
	Range	Suggested
Sand 0-2	60-70	67
Filler	10-30	14
Bitumen Grade 80/100	14-20	19

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 before actual implementation at site..

Traditional methods may be used if employment of an asphalt mixing plant can not be justified due to limited quantity of bituminous grout to be produced for the pilot project.

The following principle requirements have to be met:

- Separate and clean storage facilities for bitumen, sand and filler, so that contamination with other materials cannot occur.
- 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, and
- 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 in all stages and results are to be recorded.

The heated mineral fractions shall have a minimum temperature of 140° C and a maximum temperature of 220° C.

Bitumen, which has been heated to exceed a temperature of 190° C, 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° C, which is to be regularly controlled at the spot.

12.3 Stone Fill Rip-Rap

(a) General

"Stone" shall include natural stone material obtained from rivers or quarries.

(b) Material Standard and Tests

The following standards shall apply:

- BS 812: Testing aggregate
- ASTM C 88: Aggregate soundness
- ASTM C 535: Abrasion resistance

Following test results shall be verified:

- 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 II).
- Water absorption of rock material shall not exceed 6% (BS 812, Part II).
- Minimum compressive strength as per ASTM C 170-50: 100 N/mm^2.
- The percentage of wear shall not exceed 40% (ASTM C 535), when subjected to Los Angeles Test.

(c) Grading Tests at Point of Delivery

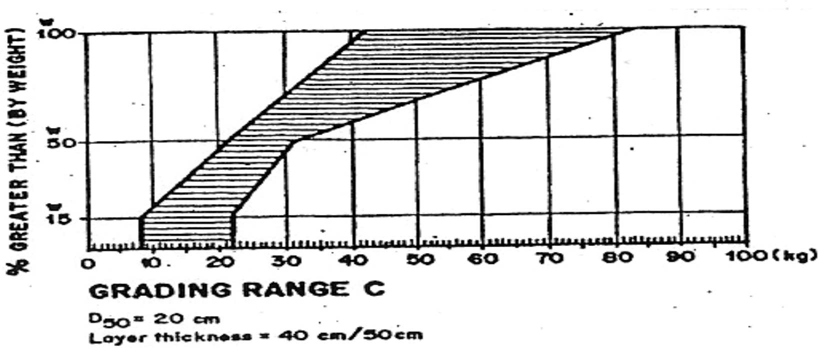
Testing at the agreed point of delivery (stockpile yard) shall take place at the supplier's expenses.

The gradation of materials stock piled at the yard shall be tested at least one time for each 500t of delivery.

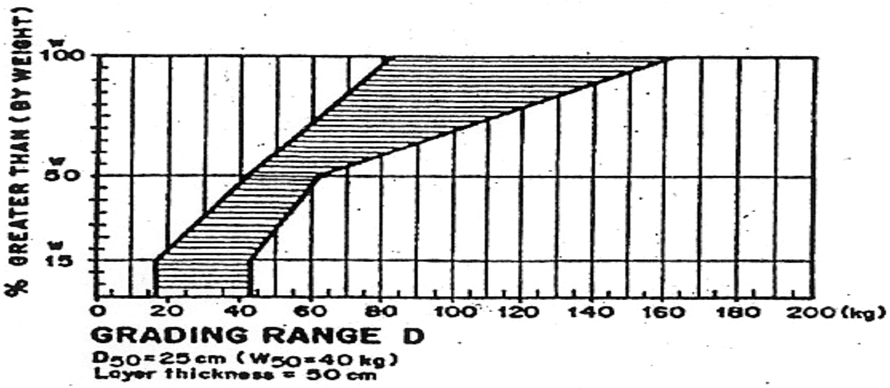
Samples for determination of weight gradation shall contain at least 100 individual stones/ rocks. The samples shall be taken by random selection from each specified gradation to obtain representative samples, and shall confirm to the grading shown in Table 12-1 to Table 12-3.

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.

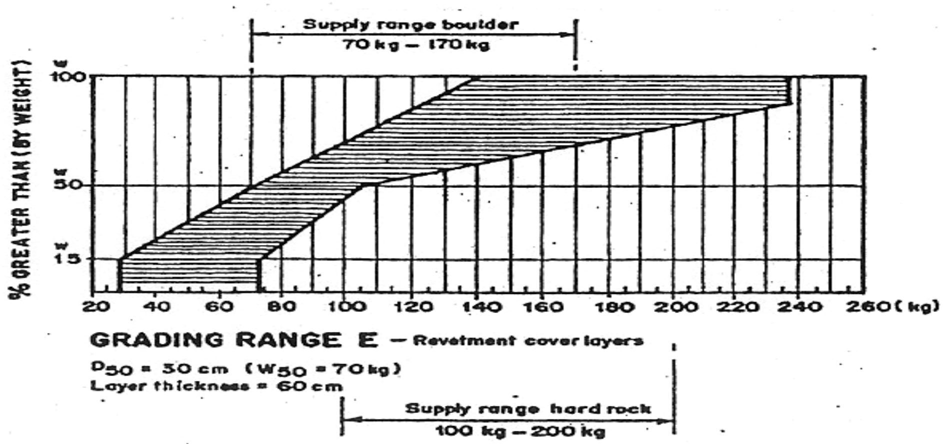
In case of non-compliance with the specified gradation range more tests may need to be performed. In case the additional tests show non-compliance the whole or part of the rock delivery may be rejected.



Rip-rap Range (a)
(suited for pattern grouting by bitumen grout)

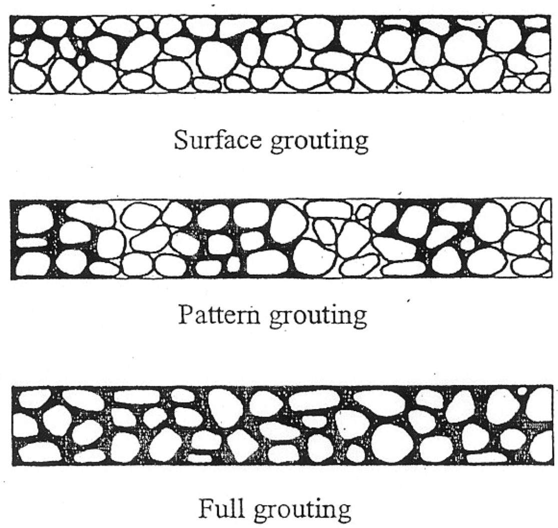


Rip-rap Range (b)
(suited for surface grouting by
colloidal cement grout)



Rip-rap Range (c)
(suited for loosely placed stones / boulders and
quarry rock)

Fig. 12.1 Riprap Grading range
(Source: Guidelines and Design Manual For Bank Protection, FAP-21, December 2001)



(Source: PIANC, 1992)

Fig. 12.2 Bonded stones grouting

CHAPTER THIRTEEN

Wiremesh Mattressing

13.1 General Description

Wiremesh cages for mattress-type protective layers are containers made of galvanized or PVC-coated steel wire mesh. The cage thickness and size is dependant on the hydraulic load and the fill material. Normally, they are filled with stones or bricks or brickbats. The size of the stones and the thickness of the mattresses can be determined using the formulas for current attack and wave attack.

Wiremesh mattresses are very flexible elements. The thickness ranges between 15 cm and 30 cm. 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.

The diaphragms must be perpendicular to the direction in which the filling will move, either down to the slope or in the direction of the flow. Prior to filling, the individual cages must be connected to each other along with all the 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.

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 preassembled 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. Due to high executional risk when working under strong river flow conditions, use of such mattresses for under water application is not generally recommended.

13. 2 Material Specifications

13.2.1 Wiremesh Cages

Wiremesh mattresses are large and thin box-type construction elements made of zinc plus PVC-coated hexagonal double twisted wire mesh (wire diameter 2 mm/3 mm including coating layers). The mesh is stretched on steel bars with a diameter of at least 12 mm, which forms the edges of the mattress cages.

Mattress dimensions of 4×2 m are recommended, with partitions at every 1 m. The nominal mesh size varies from 44×60 mm to 100×120 mm. The recommended mesh size is 60×80 mm.

The mattress thickness depends on the type of fill material and is selected to

- d = 20 cm (stone fill)
- d = 35 cm (full-sized brick-fill)
- d = 30 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, and
- PVC-coating conforms to ASTM and has a thickness of 0.5 mm.

13.2.2 Brick Fill

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.

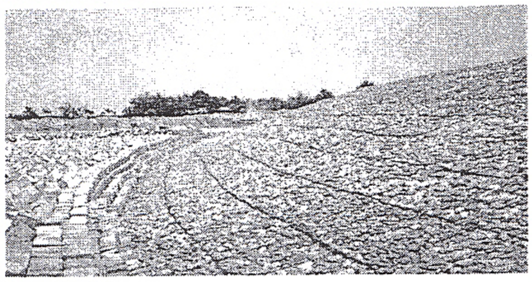
The brick dimensions shall be 240×120×70 mm, with ± 5 mm tolerance for any of the dimensions. Minimum crushing strength shall be 15 N/mm^2. The increase in weight shall be less than 16% after wetting in water for one hour.

13.2.3 Stone Fill

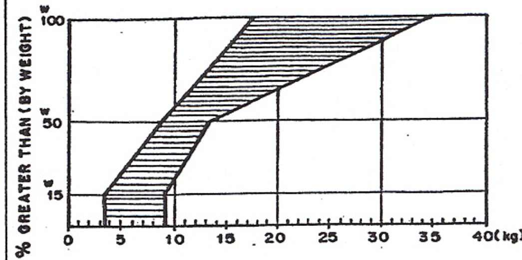
The stones shall be good sound material without cracks/fissures to avoid breaking during handling and placing or dumping.

Materials standards and tests shall confirm as said set in the standard test procedures.

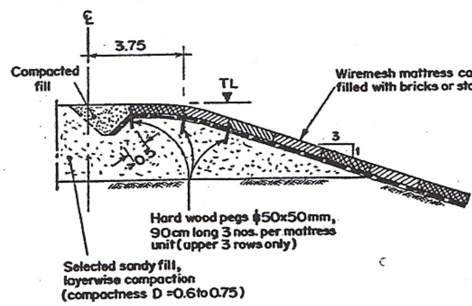
Only stone/rock with a factor not exceeding 2.5 between the longest and shortest dimension of the rock shall be used.



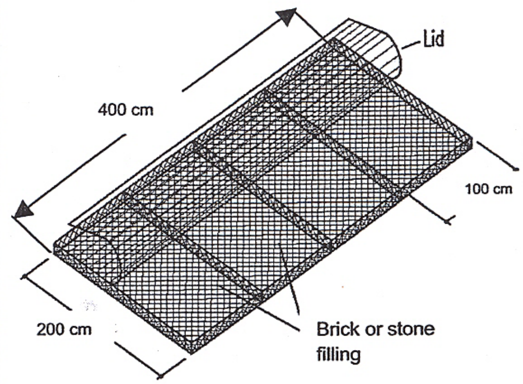
View of completed revetment with stone-filled wiremesh mattresses



Stone fill – Grading Range B – $D_{50} = 15$ cm



General detail of wiremesh mattress arrangement as revetment of upper slope zone

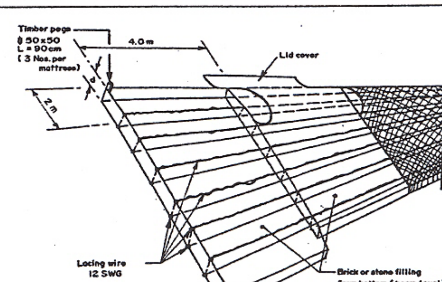


Typical design of local-made wiremesh boxes
(each compartment framed with welded ø 12 mm plain reinforcing bars)

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

Fig. 13.1 Wiremesh mattressing (source: Guidelines and Design Manual For Bank Protection, FAP-21, December 2001)

13.2.4 Grading Tests at Point of Delivery

Testing at the agreed point of delivery (stockpile yard) shall take place at the supplier's expenses.

The gradation of materials stockpiled at the yard shall be tested at least one time for each 500 tons 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 13.1 and Figure 13.1.

Table 13.1 Stone ranges for fill of wiremesh mattresses

Range B	Stone 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

CHAPTER FOURTEEN

Protection Elements

14.1 General

Protection elements form the basic units of revetment/rip-rap and toe protection. Size and weight of each unit, specific gravity/unit weight of the material, resistance against abrasion, durability against weathering action and environment friendliness are the basic factors for selecting a protection element. Availability of the material within the country, economic viability and sustainability are other guiding factors for such selection.

14.2 Boulders/Rocks/Stones

(1) Boulders / rocks shall conform to the sizes/weights and grading shown on the drawings. The material shall not be polluted, and shall be free from objectionable quantities of dirt, sand, dust and elongated or flaky stones. The ratio between the smallest and largest dimension of single stone shall generally be not less than 0.4.

(2) The boulders/rocks shall be free from cracks and veins, which could lead to breakage during loading, unloading and dumping. The bulk specific gravity of the boulder shall have a minimum value of 2600 kg/m^3 as per BS 812;

(3) Water absorption of stone/boulder shall not exceed 6% (BS 812).

(4) Minimum compressive strength as per ASTM C 170-50: 100 N/mm^2.

(5) The weighted average loss of materials in the sodium sulphate soundness test shall not be more than 10% by weight in accordance with ASTM C88. The percentage of wear as determined by the Los Angeles Test shall not be more than 40 as per ASTM C535. The aggregate impact value on average shall not exceed the 30% limit included in BS 812; Part 3, Chapter 6.

14.3 Pre-cast Concrete Blocks

(1) Pre-cast concrete blocks shall be made to the dimensions shown on the drawings and to the specified tolerances. The materials and workmanship shall comply with the standard specification in all respects. The blocks shall comply with the percentages of the different sizes as shown on the drawings. A size wise schedule of all blocks shall be prepared before execution of the work.

(2) Except otherwise shown on the drawings, pre-cast concrete blocks (cc blocks) shall be made from concrete (compressive strength ≥ 10 N/mm^2) and cast in moulds formed from steel sheet. The moulds shall be sufficiently tight fitting to prevent grout losses and sufficiently rigid to withstand the effects of pouring and vibrating during placing the concrete without distorting and capable of releasing the hardened concrete blocks without causing damages to the blocks.

(3) Each block shall be marked with a serial number and the date of casting; marking shall either be engraved on the block whilst the concrete is still "green" or painted on the

block with water proof paint immediately after stripping formwork. A register (officially issued) of the number, date of casting, date and location of placing of each block shall be maintained at site and available at all times for inspection.

(4) Blocks shall not be stockpiled until they have been cured. They shall not be placed in the works until at least twenty one days after casting have elapsed or the specified strength has been attained.

(5) Blocks, which are damaged during transport, stockpiling or handling, shall be rejected and removed from the site.

(6) Blocks for use in launching aprons shall be stockpiled in different sizes and in the percentages shown on the drawings. Prior to the commencement of placing the blocks, a framework shall be made to ensure that the different block sizes are well distributed as desired by the designer.

(7) Subject to the approval of the designer / engineer, geometry of the large blocks may slightly be modified 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.

14.4 Gabions/ Wire-netting crates

Gabions or wire-netting crates are box-type construction elements made of zinc plus PVC-coated hexagonal double twisted wire mesh (wire dia. 12 SWG including coating layers).

Dimensions of crates/gabions are generally (a) 0.90x0.60x0.45 m (b) 0.60x0.60x0.45 m, and (c) 0.50x0.50x0.45 m.

The nominal mesh size is 44 x 60 to 100 x 120 mm, hexagonal. The recommended mesh size is 60 x 80 mm.

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 and
- PVC-coating conforms to ASTM and has a thickness of 0.5 mm.

Fill Material

Bricks/boulders shall be used for filling gabions/crates

Brick Fill

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.

Stone Fill

The stones shall be good sound material without cracks/fissures to avoid breaking during handling and placing or dumping.

Only stone/rock with a factor not exceeding 2.5 between the longest and shortest dimension of the rock shall be used.

14.5 Geotextile Bags

(1) Geotextile bags shall be manufactured from short staple or continuous filament non-woven geotextile weighing not less than 0.4 kg/m^2, and with 0_{90} not greater than 0.07 mm or similar material as noted in the technical specification.

(2) Geotextile bags shall be manufactured to the dimensions and capacity specified on the drawings and filled with sand which complies with the requirements of design drawing.

(3) The bags shall 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.

(4) Geotextile bags shall be manufactured from polypropylene or polyester fabric and shall be short staple or continuous fibre needle punched and not solely thermally bonded.

(5) The properties of geotextile fabrics for manufacturing bags shall be tested according to the relevant standards in their latest version and meet the technical values of the following table:

Table 14.1 Properties of Geotextiles

Properties	Test Standard	Test values
Opening size O_{90}	EN ISO 12956	≥0.06 and ≤0.08 mm
Mass per unit area	BS EN 965	≥ 400 g/m^2
CBR Puncture Resistance	EN ISO 12236	≥ 4000 N
Tensile Strength (machine direction-MD or cross machine direction-CMD)^{*1}	EN ISO 10319	≥ 20.0 KN/m
Elongation at maximum force-MD	EN ISO 10319	≥ 60% and ≤ 100%
Elongation at maximum force-CMD	EN ISO 10319	≥ 40% and ≤ 100%
Permeability (velocity index for a head loss of 50 mm – V_{H50}	EN ISO 11058	≥ $2*10^{-3}$ m/s
Minimum thickness	EN ISO 9863	≥4.00 mm
Abrasion	Following RPG of BAW, Germany, O_{90} according EN ISO 12956 and thickness according BS EN 9641	After test: tensile strength ≥ 75% of specified tensile strength, thickness ≥ 75% of original value, O_{90} ≤ 0.09^{*2}
UV Resistance	ASTM D4355	≥ 70% of original tensile strength before exposure

(6) Geo-textile bags, filled with local fine sand, are used for riverbank protection. The bags must withstand loads resulting from filling, handling, transporting, dumping and hydrodynamic forces. It is important that the sand does not leak out over time.

(7) Each batch of bags delivered at site shall be packed in standard numbers and marked with labels that identify the (i) brand and grade, (ii) production lot number and date of production of geotextile, (iii) number of bags, (iv) size of bags, and (v) name and signature of the quality control person certifying the compliance of all bags per bale.

(8) Each bag shall be double stitched along all edges except for the opening at the top of each bag which shall be closed after filling. The minimum tensile strength of the seam shall be not less than 90% of the tensile strength of the geotextile.

(9) The number of stitch per inch should not be less than 5. The stitch shall be double thread chain type .

(10) The two lines of stitches shall be within 5 mm distance with a margin of 2 cm from the edge of the geotextile to the centerlines between the two seams.

(11) The thread used for the seam should be of same material as the geotextile (e.g. polypropylene or polyester).

(12) At the bottom end of each seam (at the folded site) the stitch shall be locked either by stitching one time back and forth for a length of minimum 2.5 cm from the end of the bag, or by joining the ends of the two threads e.g. by gluing, welding, knotting or other appropriate methods.

14.6 Porcupines

Porcupines are box like bamboo frames with leg extensions of framing bamboo units from all corners. Porcupines are ballasted by filling the box with brickbats (not smaller than ½ bricks) or boulders (not smaller than 150 mm).

Barak bamboo of dia 75 to 100 mm having effective length of 1.8 m shall be fixed by 200 mm long nails and tied with strings. The size of central chamber of the porcupine shall be 0.6m x 0.6m x 0.6m or as specified in the drawing.

14.7 Grout Filled Mattress

Grout Filled Mattress can replace the traditional revetment with CC blocks or other types of revetment normally used. The advantage of this type of mattress over others is quick installation with limited manpower.

The mattress is manufactured from a double layer of open selvage fabric joined in a mat configuration with multiple panels. The fabric is woven and made from very high strength multifilament polyester. The mattress is capable to withstand inflation pressure of 1.0 MPa without rupture of the fabric or bursting of the seams. Normal inflated thickness of the mattress is about 140 mm. The mattress is pumped full with grout consisting 350-400 kg of cement per m^3, fine aggregate and water. The 28 day strength of grout shall be minimum 20 MPa. The slurry (cement grout) is injected at sufficient pressure to fully inflate the mattress in a hard packed state with mattress thickness at the ridges conforming to the specification. In addition to the filter point around each panel, a geotextile filter is provided below the mattress to take care of drainage.

14.8 Technical Specification for Concrete Blocks

14.8.1 General

Fine and coarse aggregates shall comply with BS 882: Part 2. Testing of aggregates shall be in accordance with BS 812.

Appropriate means of storing the aggregates at each point where concrete is made shall be ensured, 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, and
- each heap of aggregate shall be capable of draining freely.

14.8.2 Cement

Cement shall be Ordinary Porland Cement complying with BS 12 or equivalent standard. A work 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 and shall be forwarded for analysis and testing to a laboratory acceptable to the owner.

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.

14.8.3 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:

Table 14.2 Grading for fine aggregate for concrete

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

14.8.4 Coarse Aggregates

Coarse aggregates for concrete blocks to be used in cover layers or falling aprons shall be natural or crushed stone. The pieces of aggregates shall be angular in shape and have granular or crystalline or smooth surfaces but not glossy non-powdery surfaces.

The amount of clay, fine silt, fine dust occurring in a free state or as a loose adherent shall not exceed 1%. The sum of percentages of all deleterious substances in any size shall not exceed 3% by weight. After a minimum period of 6 hours immersion in water, the previously dried sample shall not have gained more than 10 percent. Specific gravity shall not be less than 2.60.

Coarse aggregates shall be well graded within the following range:

Table 14.3 Grading for coarse aggregate for concrete

BS 410	75mm	37.5mm	20mm	10mm	5mm
% of passing	100	95-100	35-70	10-40	0-5

14.8.5 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.

14.8.6 Nominal Concrete Mix for Blocks

Concrete for concrete blocks shall correspond to concrete having minimum compressive strength after 28 days of each test cube: 10 N/mm^2

The mix proportions shall be 1:3:6 and the cement content shall be at least 260 kg for one m^3 of ready mixed concrete.

The free water to cement ratio shall be within 0.45 to 0.55 by weight.

14.8.7 Quality Control

During the production of concrete blocks quality tests shall be performed in accordance with the DIN 1045 or BS.

Six test cubes of 20x20x20cm shall be prepared on every day of blocks production for each $100m^3$ of concrete poured to verify the compressive strength. In case more than 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 samples 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 14.2.

14.8.8 Production

Formwork and moulds shall ensure maintaining designed shapes and block sizes. They shall be 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. Rehandling should not occur at anytime.

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.

14.8.9 Curing Concrete and Protection

Concrete shall be protected from the effects of sunshine, dry wind, running water or mechanical damage for a continuous period, until the concrete has reached at least three quarters of its 28 day strength, but not less than 10 days.

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.

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.

CHAPTER FIFTEEN

Bank Revetment

15.1 General Description

Placing of protection element/cover layer materials on the slope of embankments and shanks of impermeable groynes. shall start only after the toe protection 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. In case of granular filter layers, misplacement of the previous layer, and mixing up, washing out, disintegration and sliding are to be avoided. The method of placing need to ensure a minimum of voids with 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 blocks, except by interaction (Fig. 15.1). Thus, the stability of the cover layer is dependent on the stability of individual blocks. The size of the blocks may be determined using the formulas for current attack and wave attack.

Above the low water level concrete blocks/stones 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.

Concrete blocks/stones above low water level may also be laid by hand on the filter and adjacent blocks in the same row shall be laid without a gap between them.

Below the design low water level (toe protection), rocks, boulders, concrete cubes or geobags as selected by the designer shall be laid by controlled dumping. The nominal thickness of protection layers shall be achieved over at least 75% of the area and nowhere shall the coverage be less than 95% of that as per design.

A regular inspection, monitoring and maintenance programme of all the protection works shall be made mandatory.

15.2 Construction Sequence

(1) The revetment works shall commence from the most upstream part of the eroded bank, based on a chainage to be furnished by the Engineer.

(2) The alignment of the starting point of the revetment slope may be adjusted, subject to the approval of the Engineer, to minimize the amount of fill without compromising the stability of the existing embankment.

(3) The river bed at each section of the revetment shall first be built up to design level, the launching apron placed and then the sloping revetment constructed from the toe upwards.

15.3 Preparation of Toe Foundation

(1) The toe foundation shall be either excavated or built up to the lines and levels shown on the drawing.

(2) Excavation shall be undertaken in accordance with Article 13.2.2.

(3) Where necessary, the river bed and bank slopes below the low water level, shall be built up by placing or dumping earth-filled gunny bags in layers, working from the bank into the rivers. Building slope below low water level by any means of filling shall be avoided as much as possible. The river bank slopes above water shall be builtup with compacted earth and earthfill gunny bags if required; see paragraph (6).

(4) The eroded embankment/river bank above water level shall be cleared, stripped and then trimmed back to stable sections.

(5) After completion of the toe preparation, the filling above water level shall be commenced from the bottom of the slope. Compacted 150 mm thick layers shall be formed. A three gunny bag wide dam shall form the outer extremity of each layer, with the fill compacted up to it.

(6) Care shall be taken to form the gunny bag extremity into a neat and dense slope to the line and levels shown on the drawings.

(7) Regular soundings or levels shall be taken to ensure that either the excavation is being undertaken to the design line and levels or gunny bags are being closely packed to the correct lines and levels.

15.4 Toe Protection

Toe protection of revetments may be provided by following methods:

(1) Extension to maximum scour depth: Lower extremity of revetment placed below expected scour depth or founded on non-erodible bed materials.

(2) Placing Launchable Stone: Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack. At the estimated maximum scour depth, the launching apron is assumed to cover and stabilize the bank-sided river profile, preventing from further erosion of the bank. This method has been widely used on sand bed streams. Types of application include:

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

The method (1), as explained above, is the preferred method, but can be difficult and expensive when underwater excavation is required. ‘Trench fill revetment’ can be employed if land is available along the bank without disturbing habitations or important installations. However, in ‘Weighted Riprap Toes’ is the most desired and preferred method in most of the areas along major rivers of Bangladesh.

Weighted rip-rap method can be best achieved through falling aprons. The falling aprons and construction methods with different protection material is further detailed in Chapter-18.

15.5 Revetment Material (protection element)

The revetment material shall either be boulders, cement concrete blocks, brick mattressing, boulder mattressing or grout filled mattress. (Chapter-17)

15.6 Apron Material/Toe Protection (protection elements)

The apron material shall either be boulder, rocks, cement concrete blocks, crates filled with bricks or boulder or sand filled geotextile bags. (Chapter-17)

15.6.1 Toe Protection/Apron below DLW L

Toe protection of revetment works following the "Weighted Rip-Rap Method" can be described as follows:

(1) Below Design Low Water Level (DLWL) protection elements (cc blocks/ angular stones/ boulders/ geobags) shall be laid by controlled dumping. On the sloping face and in the area below that protection elements are to be placed to the profiles shown on the drawings.

(2) Protection elements are to be randomly placed to form the loose apron and placed in proportions shown on the drawings from conveniently located stockpiles of the individual size.

(3) Dumping of protection materials (in apron) below DLWL to the bank profile shown in the drawing is normally done by dumping quantity of protection material (assorted as per design) from barges or boats. Suggested protection material per unit area (one square meter) is dumped from the barge/boat along a strip or grid as can be identified from bankline or through marker placed in the river. This can be best accomplished by properly positioned and anchored flat top barges. The position of the barge is monitored by a RTK-GPS or Total Station. The dumping shall start from the river end of the designed apron, advances towards the bank line in one meter step after dumping the designed quantity and ends at embankment toe or bank slope up to design low water (DLW).

(4) In all the major rivers of Bangladesh, construction of apron shall be complete before the start of monsoon or specifically before 1st week of May.

(5) No construction work on under-water apron shall start if the flow velocity is 1.5 m/sec or higher. This can be relaxed only when emergency protection or repair of protection work is needed.

(6) A reserve equivalent to at least 20% of apron material shall be maintained in first year after construction to meet any emergency during monsoon. From the second year the reserve material may be 10% of apron material.

15.6.2 Revetment above DLWL

(1) Above the water level concrete blocks or stones shall be laid by hand on the filter layer. The inverted filter layers and revetment material placement shall start from the toe and progress up the slope of the embankment.

(2) The fine filter layers (for granular filter) shall be placed and lightly tamped into place, followed by the coarse filter layer which shall be sufficiently compacted to support the overlaying material.

(3) The inverted filter (if granular filter is used) shall not advance more then 1.0 m up the slope before being covered by the specified overlaying material to assist placement and prevent damage to the filter layer.

(4) Above DLWL the overlaying material shall be laid on the filter in rows parallel to the direction of the current. The blocks in each row shall be staggered half a block width from those in the row below. Adjacent blocks in the same row shall be laid without any specified gap between them. The materials (blocks or rocks or boulders) shall be laid in a manner so as not to damage or displace the underlying filter.

(5) During the placement of the blocks or revetment material, the underlying filter shall not be disturbed by removing or denting a portion thereof by any manner harmful to the filter. Any damage to the filter during overlaying shall be repaired immediately.

(6) The outer surface of the completed revetment above DLWL shall have a smooth appearance with minimal unevenness.

15.7 Repairs to Bank Revetments

(1) Selection of reconstruction lengths

Prior to the commencement of the works, the actual lengths of revetment that are to be repaired or reconstructed must be confirmed.

(2) Sequence of construction

The construction programme and the intended sequence of construction, taking full account of the need to protect the works during construction shall be formulated in detail. Consideration should be given to progressing the works as follows:

(a) removal and stockpiling of existing revetment materials;
(b) investigating and repairing minor damages in the adjacent revetment;
(c) preparing, backfilling, compacting and trimming the earth embankment;
(d) construction of launching apron and
(e) construction of the slope revetment, commencing from the toe and working up the slope.

(3) Removal of the existing revetment material

The existing revetment material shall be removed from the agreed reconstruction part and the earth embankment cleared of all debris. The c.c. blocks and any boulders shall be stockpiled and sorted in locations approved by the Engineer.

(4) Construction of Bank Revetment

The construction of the bank revetment shall be undertaken as described in previous articles.

(a)

(b)

(c)

Fig. 15. 1. Manufacturing of CC-Blocks at the construction site: (a) blocks are numbered and counted properly; (b) laying geotextile mat; (c) placed CC blocks (revetment).

CHAPTER SIXTEEN

Groyne

16.1 Impermeable Groyne

16.1.1 Construction of impermeable RCC groyne

RCC groyne has mainly two parts: the earthen shank (attached to the river bank) and the RCC part (head) in the river side. The purpose of construction of RCC groyne is, as that of Impermeable Spur/Groyne, intended to stop the river bank erosion and divert the near bank river flow towards the main river. Like all other types of groynes the RCC groynes should normally be constructed in series.

The site and length of the RCC groyne has to be selected on the basis of river morphology and erodibility of the site. The main considerations of construction of the RCC groyne are: (i) the river mainly be braided in characteristics carrying predominantly coarse sediment/bed load, (ii) existence of a char land between the bank and proposed diversion of flow i.e. a main channel, is preferred (Figure: 16.1); without this char land construction of RCC groyne is very difficult and not normally feasible (iii) the near bank chute channel is closed by construction of earthen shank (iv) the RCC part is constructed in the char land. The chute channel is closed by either with dredge filled soil or suitable construction soil carried from nearby area. The closure and thereby construction of earthen shank above design low water level (DLWL) should be completed before rising of the water level. Adequate launching material has to be provided at the u/s toe of the earthen shank at closing section, and also around the head of earthen shank.

The bottom level of the RCC wall and pile cap of the RCC head is constructed just above the DLWL (Figure 16.2). So the construction of cast-in-situ piles, pile cap, RCC wall, and beams etc upto about 1.5 m above DLWL has to be completed before rising of water level.

16.1.2 Components of RCC groyne

The components of RCC groynes are given below:

i) Earthen shank to close the near bank channel (Figure: 16.2)
ii) U/S slope pitching of the earthen shank (Figure: 16.4)
iii) Launching apron of the earthen shank (Figure: 16.4)
iv) D/S slope protection with geotextile and CC Blocks (Figure: 16.4)
v) RCC part: 150 m for the Ganges and the Jamuna Rivers which is divided into 3 parts of 50 m each, 99 m for the Teesta and Dudkumar Rivers which is divided into 2 parts (Figure: 16.5)
vi) Cast-in-situ pile RCC Head (Figure: 16.5)
vii) RCC columns, beams and 20 cm thick RCC wall (D/S side) (Figure: 16.6)
viii) Stiffeners (Extra RCC piles) (Figure: 16.3)
ix) Deflector at the u/s end of the RCC Head (Figure: 16.3)
x) Steel sheet pile under the bottom of RCC slab/pile cap (Figure: 16.2)
xi) Top slab, railing etc (Figure: 16.6)
xii) Launching materials around the RCC part and Earthen Shank Head (Figure: 16.3)
xiii) Joint between the RCC part and the Earthen Shank (Figure: 16.3)

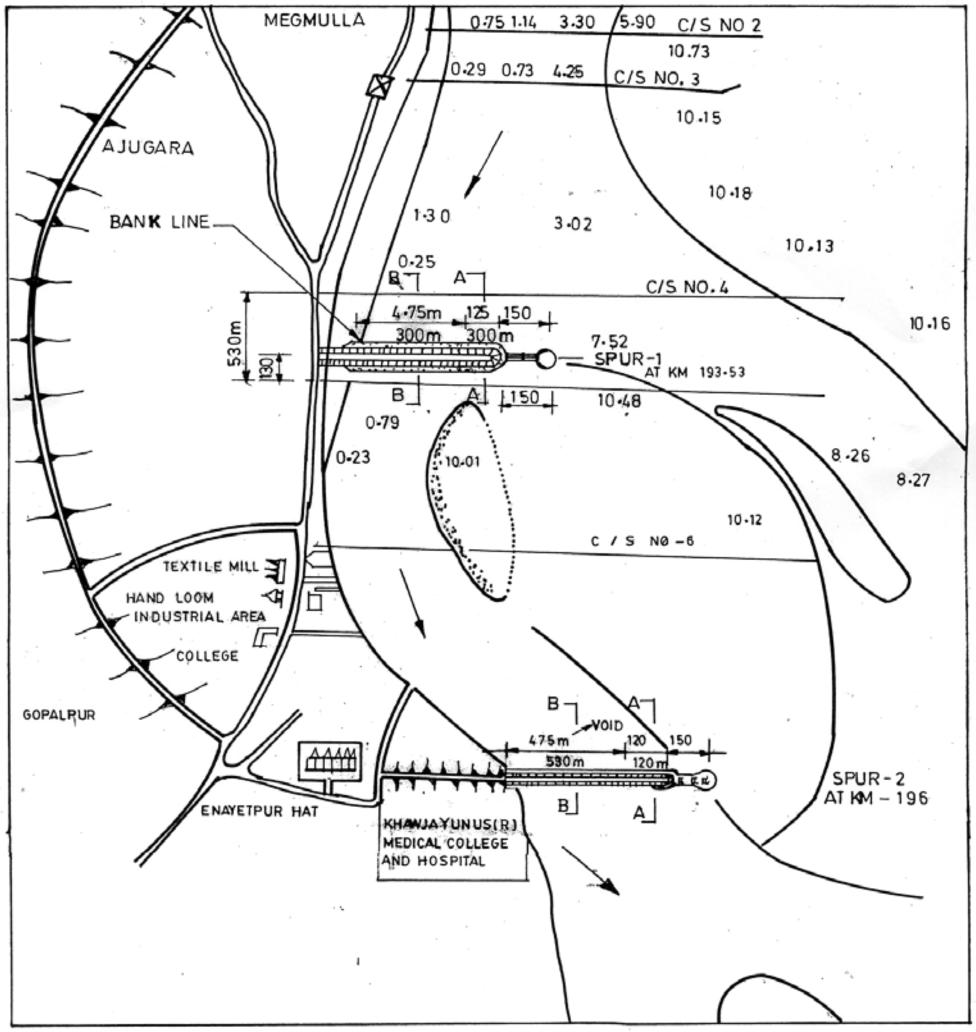


Fig. 16.1 Betil and Enayetpur Site plan of a typical RCC groyne (Source: Uddin 2007)

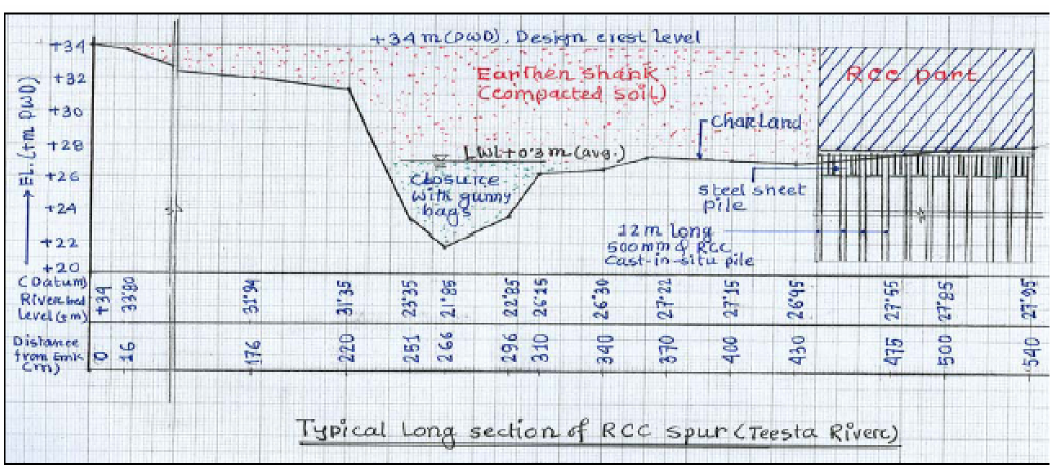


Fig. 16.2 Typical long section of the RCC groyne (Source: Uddin 2007)

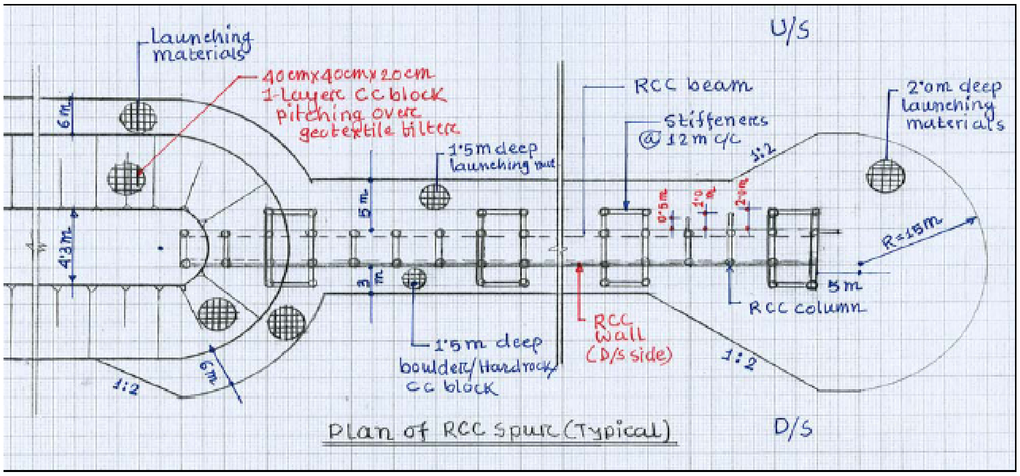


Fig.16.3 Typical Plan of a RCC groyne (Source: Uddin 2007)

Construction of earthen shank

The earthen shank closes the near bank channel and connects the RCC head of the groyne. CC blocks (normally 40cmx40cmx20cm) are used for slope pitching in the u/s part of the earthen shank. Variable quantity of launching material is provided (depending upon the river closure) at the u/s portion of the earthen shank to protect its toe and slope. To protect the d/s slope from wave and rain-cut, geotextile filter is placed under 15 cm soils. CC blocks of 45cmx45cmx45cm size for the Ganges and the Jamuna and 35cmx35cmx35cm size for the Teesta and Dudkumar are used.

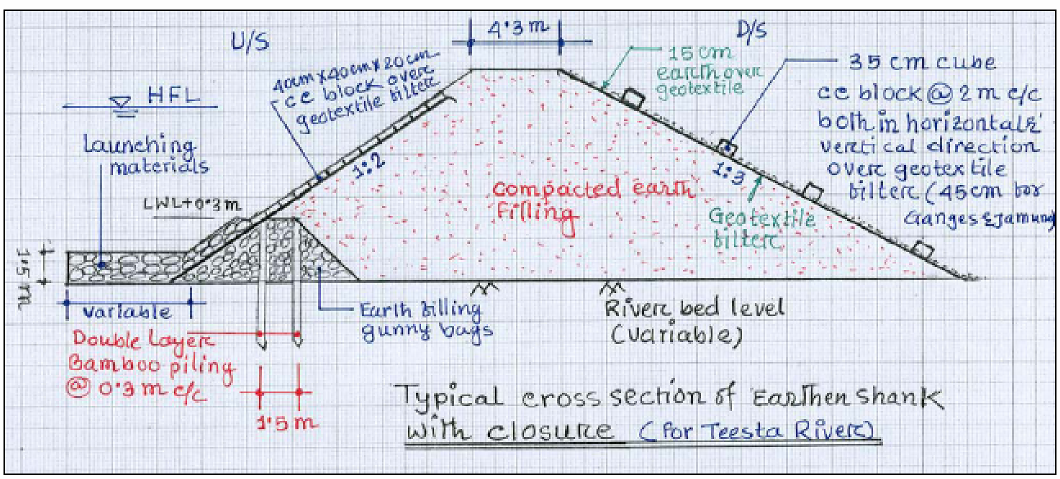


Fig. 16.4 Typical cross section of river closure (Source: Uddin 2007)

Construction of RCC part

The RCC part is constructed at the end of the earthen shank to act as head of groyne and preferably in a char land of the river.

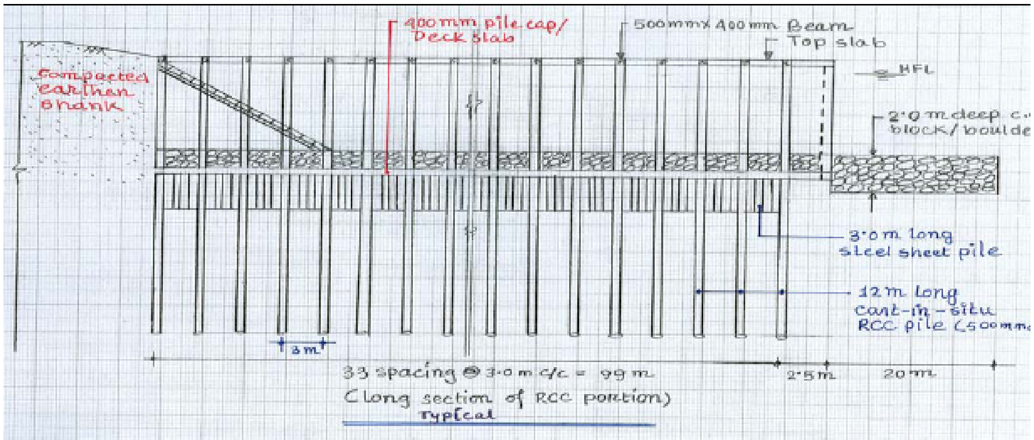


Fig. 16.5 Typical long section of the RCC groyne (Source: Uddin 2007)

The length of RCC Head measuring 150 m and 99 m have been selected on the basis of back whirling (vortex) characteristics of the rivers observed in model and prototype. A bottom slab (pile cap), on 500mm dia cast in situ piles already executed, is constructed just above the DLWL. A solid RCC wall (200mm thick) is built from the bottom slab up to the top level of the RCC groyne. Four extra RCC piles (2 at u/s & 2 at d/s) at an interval of 12m are provided to stiffen the structure against lateral thrust. CC blocks/ boulders/hard rocks are placed/ dumped along u/s, d/s and around head of the RCC wall. The volume of apron material in the form of cc block/hard rocks/boulders are calculated according to the anticipated scour depth

Execution of cast in situ piles and details of falling apron used as toe protection are described in separate chapters.

16.1.3 Construction Sequence of RCC Groyne

The maximum period available in a year for construction of bank protection works in Bangladesh is November to April. So to construct any bank protection works in a season all effort shall be made to complete the construction work in the field during this available period. To achieve that all pre-construction activities and mobilization of materials and equipments shall be completed before commencement of the actual construction work at site.

The construction activities for RCC Groyne at site shall therefore follow the construction sequence as described below having all the pre-construction activities mentioned above are complete:

(i) Execution of cast in situ piles as per design,
(ii) Simultaneous preparatory activities for construction of earthen shank,
(iii) All ancillary works for the closure portion (channel closing) is complete,
(iv) After execution of cast in situ piles in a segment (50m) sheet pile is driven,
(v) Sheet pile drive is complete after completion of execution of all cast in situ piles,
(vi) Base slab (pile cap) over the driven piles are complete,
(vii) At least one lift of the RCC wall over the base slab and all the columns is constructed,
(viii) The closure and the earthen shank is constructed upto a level of at least 1.5m above DLWL,

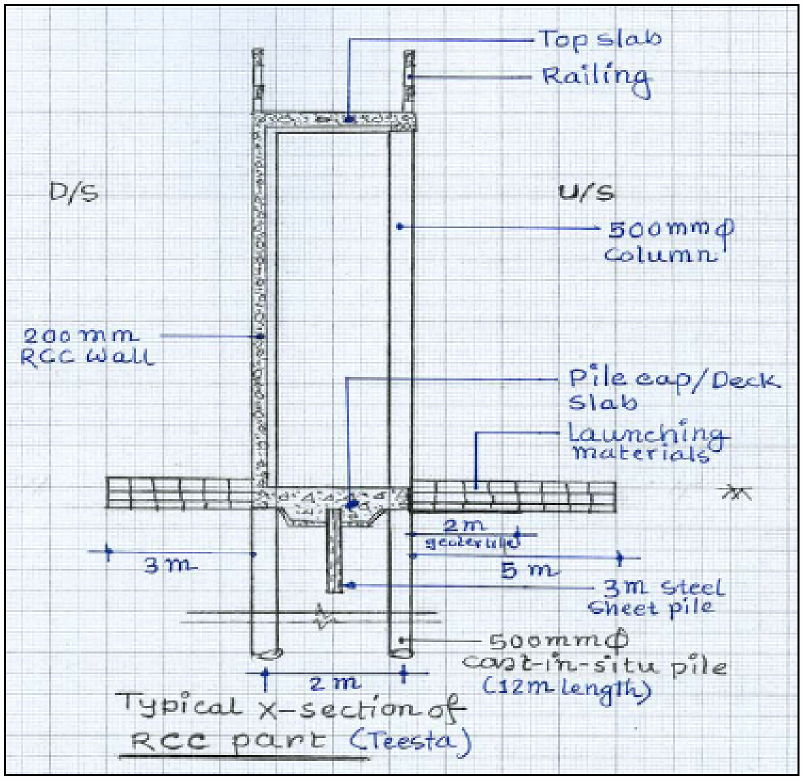


Fig. 16.6 Typical cross section of RCC part (Source: Uddin 2007)

(ix) Simultaneous progress in construction of superstructure of RCC Groyne and Earthen Shank,
(x) Placing apron protection along both u/s, d/s and around head of RCC Groyne,
(xi) Laying slope protection on filter and apron stone along u/s toe on the completed base of Earthen Shank,
(xii) Transition section (overlap) between the Earthen Shank and RCC Groyne is complete along with slope of earthen shank head upto the desired crest level,
(xiii) Laying slope protection and toe protection (apron) around the earthen shank head as per design,
(xiv) Complete the RCC Groyne upto the crest level,
(xv) Complete the earthen shank upto the crest level,
(xvi) Laying slope protection over the filter in u/s and d/s of the earthen shank with the advancement of construction of earthen shank after step (xi).

16.2 Permeable Groyne

16.2.1 General

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 impacts of river crafts. The application of timber piles is restricted to rather small channels with low water depth.

Subsoil investigation 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 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 technique.

The design of a permeable groyne must allow for an efficient reduction of flow velocities along the groyne axis, to prevent the river bank from undermining. In order to achieve that a most gradual transition from the fully blocked to partially blocked river cross-section, it is recommended to decrease the permeability of the groyne towards the embankment.

The major construction element for a permeable groyne is timber piles, steel piles or pre-cast reinforced concrete piles. The installation procedures for all such piles and also other types of piles used in bank protection works are described in the relevant chapters.

CHAPTER SEVENTEEN

Pile Installation Procedure and Equipment

17.1 Pile Categories

For application in bank protection works and erosion prevention measures within Bangladesh the envisaged materials are compiled in Table 17.1. However, the table shall not be a restriction for other materials or solutions.

Table 17.1 Pile categories

Pile Category	Material	Diameter / Size	Max. Length
Timber Piles	Bamboo bundles	< 150 mm	<7.5m
	Bullah piles	<200mm	<8m
Concrete Piles	Precast reinforced concrete	500 x 500 mm	15m (cast at site)
	Prestressed spun concrete	500mm	3@ 10m=30m
Tubular Steel Piles	Mild steel (or higher)	< 1,500 mm	No limitation
Sheet Piles (for cofferdams only)	Precast reinforced concrete	250 x 500 mm	<10m
	Special sheet pile steel	Factory standard	No limitation

The limitations presented in Table 17.1 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 10m),
- 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)

17.2 Categories of Pile Driving/Installation

This section presents some criteria for deciding the appropriate pile installation equipments; hereby, 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 method, such installation of a cast in situ RCC pile is only used in exceptional cases like RCC groyne, which is also described in later.

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. 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 also the size, weight and material of the pile to be installed. However, there are situations when the actually available pile installation equipment presents a constraint for the design engineer. It 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.

17.3 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 aforementioned 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 to minimum.

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 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.

A very rough guide on the order-of-magnitude of a vibrator capacity for steel sheet pile installation is given in Table 17.2.

Table 17.2: Guide value for determination of vibrator capacity for normal steel sheet piles

SPT N-Value	Type of Soil	Minimum Centrifugal Force per Linear Meter of Steel Sheet Pile
	Non-Cohesive Soil	
0-4	Very loose	10 KN
4-10	Loose	15KN
10-30	Medium dense	20 KN
30-50	Dense	25 KN
>50	Very dense	40 KN
	Cohesive Soil	
0-2	Very soft	10KN
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

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 (Guidelines and Design Manual, FAP-21)

17.4 Pile Installation by Hammer

The most common equipment for pile driving are simple "single drop hammers", "diesel hammers" and "hydraulic drop weight hammers".

During the execution period of the FAP 21 Pilot Project the following piling hammers were available within Bangladesh:

- single drop hammers up to 5 tons (but any weight could be manufactured, provided that the suitable pile installation tripod and which would be made available);
- diesel hammers from 0.5 ton up to 3.3 ton stroke weight, with a corresponding piling energy of about 12.5 KNm up to about 90 KNm, and
- hydraulic piling hammers of 3.5 tons, 7 tons and 15 tons with a corresponding piling energy of 42 KNm up to about 150 KNm.

Typical diesel hammer data are provided in Table 17.3.

Table 17.3: Typical diesel hammer data

Weight of Stroke (kN)	Piling Energy per Blow (kNm)	No. of Blows per Minute	Suitable for Pile Weight up to (tons)
5	12	40-60	1.5
12	31	40-60	4
22	34 – 76	38-52	6
30	44-87	38-52	8
36	52-115	37-53	10
46	67 – 146	37-53	15
55	86-160	36-47	20

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).

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. Depending on 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 17.1 presents some typical pile head packing and the respective energy reduction factors η.

17.5 Installation of Timber Piles

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.

17.6 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.

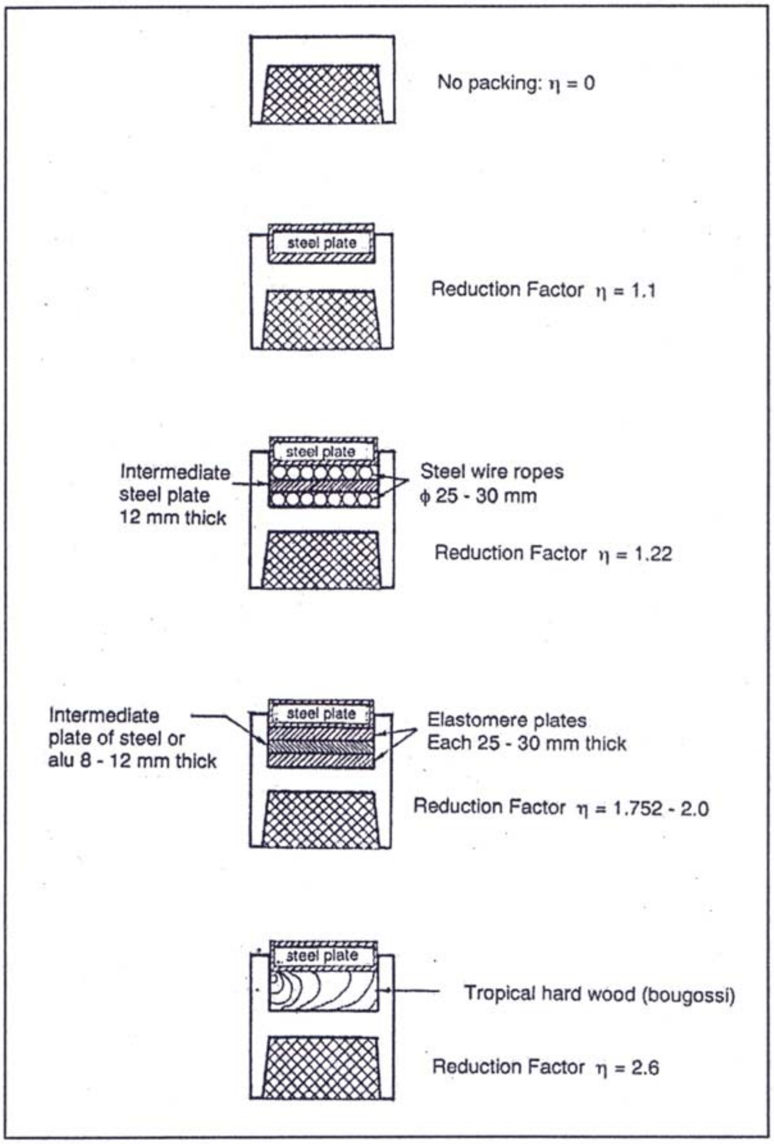


Figure 17.1: Typical Pile cap packing and related reduction factors η.

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 1.0 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 than 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 of 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 tons to 8 tons will usually suffice.

With moderate, i.e. not too dense soil conditions concrete piles up to diameter 500 mm and length of 20 m may also be driven by middle size diesel hammer.

17.7 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 diameter up to 1220 mm and a pile length of up to about 45.0 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 1.0 m is a recommended solution. At the Jamuna Multipurpose Bridge steel piles of 68.0 m to 86.0 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 l,200mm 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 utilizing 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 3.0 m to 5.0 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.

In the following section, some basic elements in order to plan the sufficient pile installation equipment are provided. Again, these are suggestions from engineering judgment, 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.

17.8 Estimation of Piling Hammer Capacity

(a) Estimation of Drivability 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 17.4 and Table 17.5.

Table 17.4 Characteristics of non-cohesive soils

CPT: q_c [MN/m^2]	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 17.5 Characteristics of cohesive soils

CPT: q_c [MN/m^2]	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

With the chosen pile type and the classified subsoil, skin friction (ζ_{mf}) and pile tip resistance (σ_{sf}) for static loading can be estimated using the general assumptions compiled in Table 17.6. 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.

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

$$W_e = U.l_e.\tau_{mf} + A_t.\sigma_{sf} \qquad [KN] \qquad (17.1)$$

Table 17.6 Subsoil characteristics versus drivability of subsoil

Drivability	Subsoil	τ_{mf} (kN/m^2)		σ_{sf} (MN/m^2)	
		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

(b) 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 We, 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: $F = U \bullet t_{min} \bullet f_y$ [KN] (17.2)

Using formula (Eq. 17.3) 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, than 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: $t_{\min} \geq \dfrac{W_e}{(U.f_y)}$ [KN] (17.3)

The finally chosen wall thickness $t_c > t_{min}$ 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_c:

Maximum permissible driving force: $F_{max} = U \cdot t_c \cdot f_y$ [KN] (17.4)

(c) 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.

Table 17.7 Initial approximation of piling hammer type and weight

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 /l to 1.5 / 1	1 / 1 to 2 / 1

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.

(d) Typical Hammer Data and Pile Cap Factors

Table 17.8 and Table 17.9 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. For hydraulically operated piling hammers the drop height of the ram can be pre-set as per requirements, ranging usually from about 15-20 cm to about 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 dependant 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 lead. 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.

Table 17.8 Technical data of DELMAG diesel hammers with hit dispersion

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 17.9 Technical data of MENCK hydraulic drop hammers

Type MHF	Weight of Ram	Weight of Pile Cap	min. Energy	Free Drop, max. Energy	Free Drop Static 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

(e) Estimation of Pile Penetration per Blow

With the initial pre-selection of a piling hammer type/capacity as per Table 17.7, Table 17. 8 and Table 17.9 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.

B1ow Velocity: $$v = \sqrt{2.g.H} \quad \text{m/s} \tag{17.5}$$

Shock Wave Velocity: $$a = \sqrt{\frac{E_M .g}{\gamma}} \quad \text{m/s} \tag{17.6}$$

Impedance of Driving Element: $$Z_p = \frac{E_M .A}{a} \quad \text{[KN-s/m]} \tag{17.7}$$

Compression of Driving Element

During Blow: $$\Delta L = \frac{M.\nu}{Z_p} \quad [\mathrm{m}] \tag{17.8}$$

Length of Shock Wave: $$L_w = \frac{\Delta L.E_M.A}{0.5.P_{\max}} [\mathrm{m}] \tag{17.9}$$

Pile Penetration per Blow: $$s = \frac{\Delta L.(p_{\max} - W_e)^3}{2.P_{\max}^2.W_e} \quad [\mathrm{m}] \tag{17.10}$$

The symbols are explained in Section: 17.8 (g)

It has to be acknowledged that the piles for permeable groyne structures are horizontally loaded elements. It is important to be aware that pile should 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 life time of such structures, it is unavoidable to have excessively long pile 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 (17.10), and as a rule of thumb the pile penetration per blow may be considered in a range of about 1 mm.

(f) 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: $$W = \frac{E.R}{(s + 0.5.c).(R + Q)} \quad [\mathrm{KN}] \tag{17.11}$$

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

$$W > W_e$$

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.

(g) Notation Used

Within the forgoing sub-chapters several notations have been used, which are compiled here again in alphabetic order for case of reference:

A [cm^2]	material area of driving element
A_t [cm^2]	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
EM [KN/m^2]	modulus of elasticity of driving element
F [KN]	permissible driving force
f_s [MN/m^2]	skin friction for Penetrometer (CPT)
f_y [N/mm^2]	yield stress (steel)
g [m/s^2]	gravity acceleration
H [m]	drop height
I_c [-]	consistency index (of cohesive soils)
L [m]	length of pile
I_E [m]	embedment length (of piles)
M [$KN\text{-}s^2/m$]	mass of ram
P_{max} [KN]	static stroke of hammer
Q [KN]	weight of pile and pile cap
q_c [MN/m^2]	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)
W [KN]	ultimate load, i.e. driving resistance of pile
W_e [KN]	estimated driving resistance (of piles)
γ [KN/m^3]	specific weight of driving element
σ_p [MN/m^2]	prestress (in concrete piles)
σ_{sf} [MN/m^2]	pile tip resistance (of piles)
τ_{mf} [KN/m^2]	skin friction (of piles)

17.9 Pile Installation Works

17.9.1 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.

17.9.2 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.

17.9.3 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 misalignment 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 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. (Figure: 17.2)

If a sheet pile or an interlock breaks or a major deviation is observed during its installation or if the designed location is not achieved or driving has not been carried out as per design, additional measures for rectification 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.

17.9.4 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 or 35 N/mm^2.

Special and tight fitting driving caps shall only be used. 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 packing 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.

17.9.5 Pile Butts

(a) 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.

(b) 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 LW 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.

17.9.6 Surveying and Tolerances

The piles shall be set up at locations as per 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 fixed 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

17.9.7 Observations During Installation

(a) 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 trail holes, and which cannot be penetrated with the appropriate equipment as required.

Prior to 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.

(b) Observations of Pile Behavior

If during installation of a pile the pile behavior 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 behavior are to be investigated and suitable countermeasures be implemented.

(c) 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.

(d) Damages to Piles

If 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.

17.9.8 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 number, 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 al 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 maintained.

17.10 Bored Concrete Piles (In-situ Concrete Piles)

17.10.1 Definition

Bored piles within the meaning of this Section are in-situ concrete piles, 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 c9ncreted. 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.

17.10.2 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.

Only experienced and reliable foremen, who have already successfully executed such work, shall supervise bored pile works.

The responsible construction superintendent must be thoroughly acquainted with the type of bored pile construction and its execution.

Piece-work shall not be tolerated by the Employer because of the therewith connected sources of danger for the quality of the bored pile construction. The same holds good for night work, which will be approved by the Employer only in exceptional cases.

17.10.3 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.

17.10.4 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 1, 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 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 mm. The water-cement ratio shall not exceed 0.60.

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/mm^2
- Minimum average compressive strength of each series of 3 cubes: 40 N/mm^2

17.10.5 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.

Bored piles shall be sunk to design depth as indicated in the Drawings and Bill of Quantities or as otherwise directed by the Engineer.

If soil layers are encountered in which technically sound construction of the concrete piles and their load bearing capacity appears doubtful, then immediate corrective measures shall be planned after further investigation of soil properties. Any further work or measures shall be carried out only after thorough investigation of soil properties.

The deeper seated piles must be constructed first if piles of different depth are proposed for a specific construction. No exception to this general principle shall never be allowed 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 bored 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.

The bore pipes must be adequately supported and secured during the entire bore pile construction, until the concrete has achieved adequate strength.

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, 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 possible to 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, those bore holes shall not be accepted for construction of an in-situ concrete pile and must be abandoned.

A continuous water supply to the bore pipe to ensure an adequately high water table in the bore hole at any stage of boring shall be maintained during drilling below water table, i.e. during drilling and on withdrawal of the drilling tool.

The excess pressure inside the bore pipe shall be at least 1.0m. 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 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 step is of particular importance if sands tending to flow or silty or loose cohesive soils which quickly change their consistency during drilling under water influx are encountered during drilling.

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.

17.11 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/specified equipment nor derived from the data provided after soil investigation.

The Engineer and the designer shall be informed immediately when obstacles are encountered during drilling. All subsequent measures shall therefore be coordinated with the all of them.

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.

17.12 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 neighbouring 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. In any case, prior to concreting of such boreholes, it must be observed whether the bottom level of the bore point is lifted within that time period. Under certain circumstances, re-boring must be carried out before 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 bore pipes shall be withdrawn slowly and uniformly, particularly in the upper length of the pile, to avoid breaking or contracting of concrete column. During extracting of bore pipe, 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.

17.13 Records of Installation

Detailed records on the construction of every bored pile shall be maintained.

The form of reports shall meet the requirements of DIN 4014 and must be coordinated with the Employer.

17.14 Controls and Acceptances

The concrete consumption for each in-situ concrete pile must be verified and recorded. 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.

Bored piles, for which the foregoing verification report is not available, shall not be considered as permanent structural members. In such cases, a replacement pile shall be installed.

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 17.10

Table 17.10: Required strength of concrete for bored piles

Class	Characteristic Strength (minimum compressive strength f each test cube) PW28	Series strength (minimum average compressive strength of each series of three cubes) PWm
B25 B35	25N Imm^2 35N Imm^2	30 N/mm^2 40N/mm^2

CHAPTER EIGHTEEN

Falling Aprons

18.1 General Description

Falling apron is a multi layer system of protection element placed on a sloping or horizontal surface as protection against scour. The individual units are rearranged freely with 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. Apron is intended to launch when the underlying sand/sandy soil is scoured by the river current so as to form a continuous protection below the water level.

The size and volume of material may be determined using the available formulas in the Design Guideline (Part I).

When placed in dry condition, i.e., on an excavated surface just above Design Low Water, the volume of falling apron material can be well controlled by counting the number per area.

Placing of underwater apron materials require a considerably higher skill, equipment demand and standard of control.

Below the water level, falling apron material shall be laid by controlled dumping (considering a nominal void ratio of 35% for hard materials).

The nominal thickness of a falling apron layer shall be achieved over at least 75% of the area and the coverage shall be about 100% of that as per specification.

Quarried rock and boulders have better launching characteristics than concrete cubes. Again in certain experiments it has been observed that geo-bags perform better than concrete cube in launching. Since quarried rocks and boulders of required sizes are not readily available in Bangladesh in abundant quantities and concrete blocks are also expensive, so alternate material e.g., geo-bags filled with sand may be selected in certain region to be used as a launching material.

For standard bank protection structures, concrete blocks are recommended above Design Low Water Level (DLWL). Below low water level (LWL) different materials (e.g., rocks, boulders, concrete cubes or geo-bags) may be used in launching apron. However, care should be taken while using geo-bags in the falling apron.

18.2 Quarried Rock/Stone Boulders

Dumped riprap/apron is graded stone dumped on a prepared slope/bank slope in such a manner that segregation will not take place. When placed under water either as riprap or as apron stone, it shall be dumped from floating excavators or cranes with camshell, dump or split barges etc. End dumping from trucks down the riprap slope causes segregation of the rock by size, reducing its stability, and therefore, should not be used as a means of placement.

Whilst dumping stones under flowing water, they may be displaced downstream by the distance L:

$$L = 0.25hUD^{-0.5} \text{ [m]}$$

Where h is the water depth in m, U is the mean flow velocity (averaged over a vertical), in m/s and D is the nominal stone diameter in m.

Stones, dumped under flowing water, may be segregated, therefore it should be released as close to its final position as possible. Stone can be dumped by machine on slopes not exceeding 1V:2H.

18.3 Cement Concrete Blocks

(a) Material

Production of cement concrete blocks shall follow the general descriptions and specifications following standard procedure.

(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 is defined by the number of blocks to be placed within an area of 100 m^2. This will facilitate proper control of material placement.

Fig. 18.1 shows CC block dumping sequences for falling apron.

18.4 Geotextile Bags

(a) Material

Geotextile filter material shall be used for tailoring geobag to be used in falling aprons. General descriptions are presented under "Geotextile Filter Materials".

The type (quality) of geotextile to be selected depends on the scheduled filling volume of a single unit. Manufacturer’s data sheet and design criteria shall be the guiding factor for selection of proper type of geotextile fabrics.

Quarried rocks and boulders have better launching characteristics than geobags, but geobags were found to launch better than concrete cubes in a comparative test.

In a model test and prototype observation made in JMREMP it has been observed that geobags in thinner, longer horizontal apron with small pile at outer end produce more uniform launching with no vertical face, and keep scour away from the bank. Therefore in using geobag as launching material the method of dumping and thereby attaining uniform coverage in the designated area shall be the guiding factor.

(b) Dimensioning

Contrary to blocks or boulders, sand-filled geobags 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 geobags should be adequate to stabilize the eroding bank. The correct size and volume should be determined following similar procedure as traditional hard materials.

Fig. 18.1 Local method of dumping CC blocks using country boats: simple flags are marking the dumping area.

Giving consideration to the fact that no protection has practically moved under the hydraulic forces and wave loads, the equivalent weight of one protection unit can roughly be determined by using Design Formula. The corresponding sizes are given in Table 18.1.

Table 18.1: Sizes of Concrete Blocks and Geotextile Bag

Concrete Block Size Dn [cm]	40	45	50
Geotextile bag, Equivalent weight in Kg	125	180	250

(c) General Specification

General specifications for geo-textile fabrics and geo-textile bags are described in Chapter 11.

18. 5 Selection of Construction Methods

Construction of bank protection structures always require typical and previously successful construction methods with deployment of standardized equipment. Nevertheless, for any construction the Engineer shall be responsible for selection of appropriate construction methods. Within this context, the agency responsible for the construction (contractor) has to decide on the method and sequence of the work execution and has to select the type and capacity of equipment to carry out the works in accordance with the Technical Specifications and the Drawings.

The most feasible design of bank protection can only be arrived at, if the method of construction is pre-determined, well in consideration of the local conditions and the applicability of the design to the use of local construction capability and experience.

18.5.1 Construction Methodology

The tender document should contain a construction methodology which the Contractor shall agree in the contract agreement or a work methodology is proposed by the Contractor which has been agreed by the owner. As per agreed methodology the contractor is responsible for mobilization, operation and maintenance of mechanical equipment, survey and diving equipment and procurement of construction materials e.g. geobag, sand, cement, shingles etc necessary to complete the work.

18.5.2 Construction Equipment and Materials

Through the contract agreement and the work methodology the contractor should be made responsible for mobilization, operation and maintenance of mechanical equipment, survey and diving equipment and procurement of materials necessary to complete the works. The materials and equipment needed to complete a particular stretch of the bank protection work shall be ready at site before starting the dumping operation.

The type of machinery and equipment needed for bank protection works are:

- Flat top barges
- Pontoons
- Tug boats (400 HP)
- Drum mooring winches with ropes and anchors
- Crane (40-ton capacity)
- Topographic and bathymetric survey teams with equipment
- Diver team
- Sewing machines (if geobag is used as apron material)
- Generators
- Laboratory for sand and concrete strength testing
- Motorized country boat
- Concrete mixers

Number of equipment required for the construction shall be decided on the volume of work and time allowed for completion of the construction. In every such case the normal protection work shall be completed within the construction window available on hydrological condition of the river.

18.5.3 Methods for Construction:

The toe protection of a revetment construction can be achieved through two different method, these are Mass Dumping and Arial Coverage. Construction of apron may be materialized by mass dumping when an emergency protection work needs to be in place. In normal course arial coverage for construction of apron is an ideal way.

18.6 Riverbank Protection by Mass Dumping

In Mass dumping concept, a falling apron is developed along the toe of the embankment or riverbank by dumping a designed quantity of apron material within a short length along the river section.

Flat top barge equipped with front-loading ramp may be used in river section having moderate flow velocity. The barge is positioned perpendicular to the riverbank with loading ramp (part of the barge) lowered on the bank. Workers use the ramp for direct loading of bags/apron material from shore onto the barge. Bags stacked on the barge in units per meter of bankline, as per design, is counted and dumped into water on the upstream side. Once horizontal falling apron up to designed low water level is formed, with specified units dumped, the barge is moved downstream by 1 m by releasing ropes; one anchor rope in the river and one dead man (ground based anchors made of two wooden logs driven into ground with one crossed wooden log buried under ground) on the shore.

On river sections with high flow velocity it may not be possible to keep the barge in perpendicular position to the bank as above. However, flat top barge can be held length-wise in position with the current and parallel to the bank. Barge placed in this manner is kept in desired position with respect to the bank with ropes and two dead men.

Two ramps made from bamboo and wooden logs spanning the bank with the barge may be used for transporting apron materials/bags from stacks on the shore to the barge. The width of the barge should be higher or should at least match the required width of falling apron. The available work front should be sufficient for loading the barge with stacks of geobags/apron materials and dumping afterwards.

To achieve a better quality control and to ensure the design quantity per meter of bank line, the apron materials/geobags shall be stacked according to type and design quantity per meter of bank line on the shore. After counting manually, these are directly dumped into water from the upstream side of the barge. The formation of falling apron shall constantly be checked from the barge for depth and width.

When all the designed units of apron material are dumped then the barge is moved downstream by 1 m.

Total Station is used for positioning the barge while a marked bamboo stick may be used to verify location of SAL (Structural Alignment Line/DLWL) and to check formation of horizontal falling apron at designed low water level (DLWL).

18.7 Riverbank Protection by Arial Coverage

The operation involves dumping of bags along the river bank from pontoon/flat top barges at a rate of designed apron material per square meter area, to obtain a uniform cover up to the desired width of underwater strip extending from structural alignment line (SAL i.e. Design LWL) into the river channel. By employing adaptive design approach the dumping distance (from SAL) may vary with the cross-section of the river at that position, with a falling apron to cover about 10 m scour depth anticipated at the outer end. The construction design shall be finalized on the cross-sections taken immediately before execution of the protection work. Fig. 18.2 to 18.4 shows different activities for construction of geobag falling apron.

Equipment for the Arial Coverage (AC):

- Two flat top barges (dumping pontoons) minimum size11 m x 30 m joined together with steel ropes with sufficient stacking and dumping spaces on both barges and covering at least 50 m work front.
- Tugboat for towing the dumping pontoon to the approximate position in the river.
- A barge mounted 40-ton crane, for lifting and positioning river anchors within 200 m upstream and downstream, propelled by an attached tug boat.
- On the landside the barge is secured by dead man about 150 m upstream and downstream.
- Motorized country boats for transporting apron materials/bags from shore to dumping barges.

Once the dumping pontoon/barge is towed to position and anchored in the river channel, mechanical double-mooring winches are used for its movement toward bank line and detail positioning as per design. Total Station is used for exact positioning of the equipment in parallel steps from riverside toward the bank maintaining exact northing positions along upstream and downstream limits. Country boats equipped with Diesel engines are used for carrying and delivery of apron materials/cc blocks/boulders/bags from stack yard on land to barges anchored in the river. Bags from the boats are stacked along the bank side edge of pontoon/barge as per design quantity to obtain the design coverage per square meter of river bank/channel. The apron materials/bags are dumped into the river on command after the Total Station operator verifies and record exact position of dumping. Dumping of bags continues from riverside toward the bank.

After completion of dumping at one position from river side to SAL, the equipment is moved to the riverside in the upstream direction or in downstream direction, whichever is found more efficient and easier to execute.

During the execution of AC, positioning of barges and width of AC at that position shall be determined after examination of at least two riverbank cross-sections and corresponding layout map of the bank with contours printed in scale 1:500. The cross-sections used should be taken 2-3 days before starting the work.

Uniform bag coverage of riverbank (slope and bed) under fast flow current and higher depth of water is confirmed by divers' inspections.

Due to shallow depth the dumping from pontoon may be difficult. At this stage dumping may to be done from country boats, as barges could not enter into shallow depth.

The pontoon (minimum length 30m) position shall be monitored by RTK-GPS or Total Station up to an accuracy of ±10cm, and a minimum 30m long barge/pontoon should cover at least an area of 25m x 45m per working day including the provisional falling apron, as per design and technical specification.

The following points are recapitulated to execute bank protection works through Arial Coverage:

(1) Sufficient number of pontoons or flat-top barges for the timely completion of works shall be arranged at site.
(2) Total material required to be dumped in a particular stretch (proposed length of bank line to be protected) shall be stacked on flood plain/bank before start of dumping in the reach.
(3) CC blocks/boulders/sand filled geobags/apron material from stack-yard will be transported to the dumping barges/pontoons by engine boats or self propelled barges.
(4) No dumping in apron under arial coverage shall be allowed without properly positioned and anchored dumping pontoon/flat-top barge.
(5) Any strip of underwater protection from the river-side edge of the falling apron to the edge of shallow water (Average Low Water or SAL) must be completed in a day's work.
(6) The quantity of material to be dumped per unit length of bank shall be stacked along the dumping edge of the pontoon/barge and dumped at proper position as directed by the control through total station on the bank.
(7) Dumping aid shall not have any sharp corners or edges or any other features that could damage the bags or reduce the properties of the bags making them unsuitable as protective launching element.
(8) Immediately after mobilization of equipment, preparation for anchoring of pontoons should start. Anchor points on flood plain should be free from flooding and risk against wave erosion. Sufficiently strong anchors, piles, bollards or winches shall be used and safely installed.
(9) Anchor points in the river may consist of anchor pontoons equipped with winches temporarily positioned at site to hold and move the dumping pontoons during dumping of bags. Alternatively, dumping pontoons can be directly anchored into the riverbed. Standard anchors can be used, such as stockless anchors having a holding power of about 3 to 5 times the own

weight. In selecting type of anchors, flow velocity, bed material etc shall be taken into consideration.

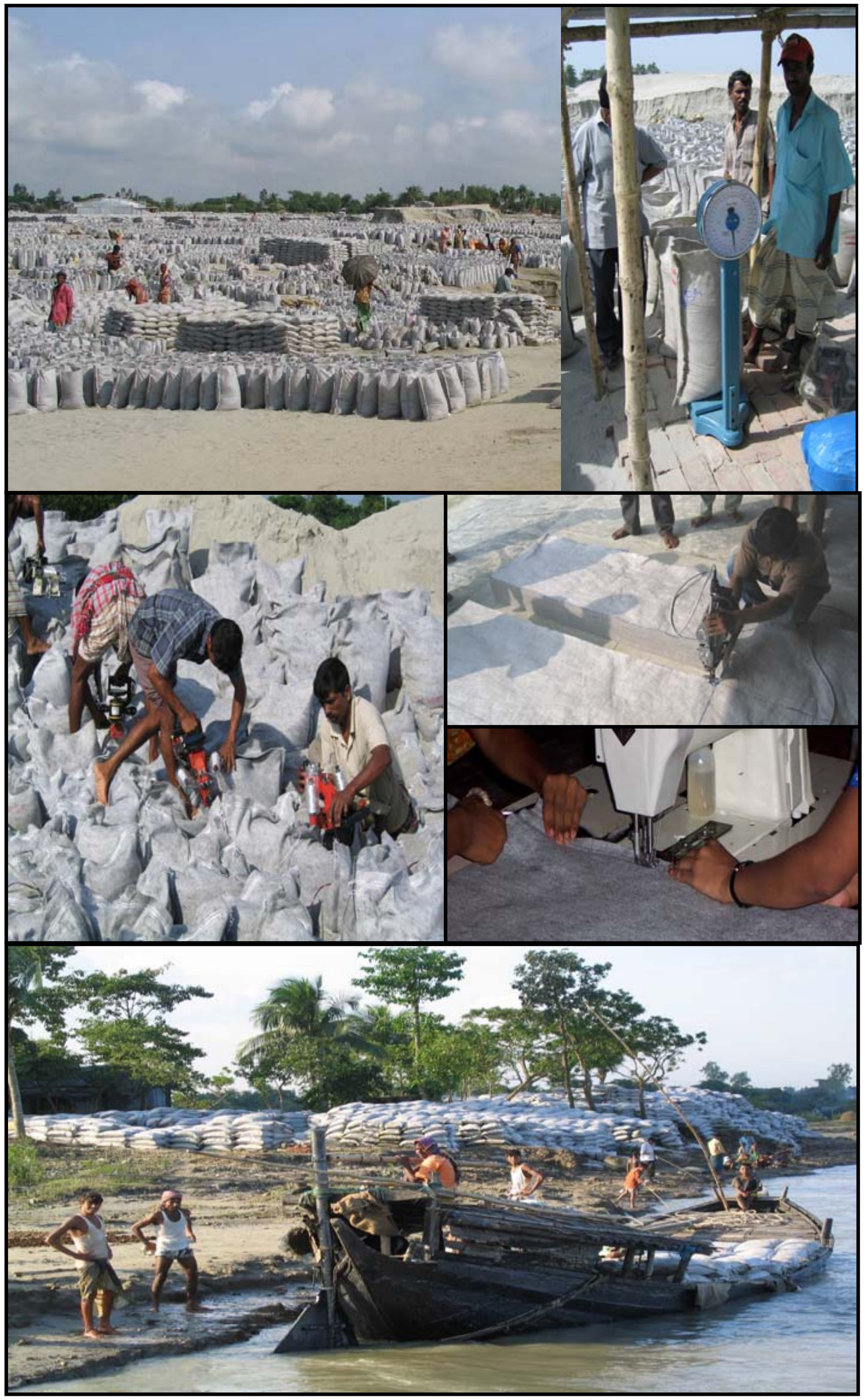


Fig.18.2 After filling and closing, bags are stacked along the bankline; bags are loaded on country boat for transport to the dumping pontoon.

Fig.18.3 The bags are loaded from the country boat onto the dumping pontoon (top); one of two double drum winches for anchoring and positioning of the dumping pontoon (middle); the dumping pontoon starts from the river and works towards the shore (bottom).

Fig. 18.4 The surveyor positions the dumping pontoon at the locations as per implementation design (top); Bags are lined up for dumping, this systematic stacking facilitated counting and recording (middle); Bags are dumped from barges in a systematic manner (bottom picture).

Records of each barge position with starting and finishing time and date, and number of bags dropped at each position are kept for future reference. All GPS positions of the barges presented are stored in memory of Total Stations and are available on soft and hard copies.

Successful protection of the river banks depends on launching the apron materials down the bank slope to the deepest scour levels, which are as much as 35 meters in depth.

To cover a 20m scour depth, apron length according to formula is: 1.5x20m = 30m,
For a 0.61m thick cover over the scoured face, material required on 1V:2H slope is (202+402)0.5x0.61= 27.28 m^3/m.

Volume of launch material per m of river bed (horizontal) = 27.28/30 = 0.91 m^3.

When considering geobags as apron material with 126 kg bags (unit volume = 0.084 m3), no of bags required per m^2 is 0.91/0.084= 10.83 nos.

18.8 Quality Control

The overall quality control shall be achieved through full time presence of Construction Supervision Engineers checking every construction activities and materials delivered to the site for compliance with the contract specifications. Boulders , sand for filling bags and concrete aggregates for CC Blocks should be tested regularly at the laboratory set up at the site. Slump tests shall be carried out twice daily and test cylinders for concrete crush strength are taken as specified and tested at a specified laboratory.

Compliance with design specifications for geo-textile bags to be checked as follows:

Sand for filling bags	: sieve analysis of delivered sand for calculating FM and test for clay and organic matter content (1 sample per 50 m^3 of stacked sand)
Bag filling with sand	: regular visual checking and weighing of sand filled geo bags
Bag stitching	: regular inspection of seams and thread
Number of bags used	: counting of stacked bags before loading on boats or pontoons; bags are stacked on the bank in number per meter of bank line proposed in design;
Quality of geo-textile fabric and Thread	: samples of geo-textile fabric and thread shall be tested following standard procedures.

Survey boat equipped with sonar scanner and divers for direct underwater inspection shall be used throughout the construction period as part of planned quality control of the protection works. Any work deficiencies like spots not covered with bags in riverbank/channel marked by divers and recorded with DGPS, shall immediately be covered with extra bags.

18.8.1 Under Water Diving Activities

The diving services are required for vulnerable sites of important river bank protection works.

- for the verification of under water works.
- to verify specific uncertainties arising from inconsistence of cross sectional surveys.
- to clarify uncertainties from side scan sonar investigation.
- to clarify disputes between contractor and engineer.
- as regular verification of the quality of the works.

18.8.2 Records

Standard forms shall be developed and used for recording all daily activities, equipment used and all types of works carried out at the site.

CHAPTER NINETEEN

Monitoring and Maintenance

19.1 General

Monitoring of river behavior and reaction of protection structure shall be the core duty of the personnel having the responsibility of maintenance of any bank protection structures. In normal structures, monitoring refers to observe and record more or less known parameters which are checked against preset and known standards. Unlike such customary structures, monitoring plays a different role in case of bank protection structures implemented in major rivers of Bangladesh. Monitoring and maintenance activities in this case can play an important role for improvement of design and construction methodology in later stages. Monitoring has to be accepted as a continuous process until a cost effective and sustainable solution for bank protection is achieved.

In order to record the response of the river with the bank protection works and also the morphological changes afterwards systematic and well documented monitoring of the river behavior is required both during the implementation and also during operation of the structure. Benefits of regular monitoring for river developments and structural behaviour can generally be:

(1) Assures that no surprises occur in terms of sudden erosion of the embankments. In this respect not only the high flood levels, but also the sudden draw down of the water levels is dangerous due to the reduced stability of saturated banks. The monitoring data obtained during the flood season provide a valuable basis for the planning of future protection work.

(2) Helps avoiding failures of bank slopes and protection embankments from potential geotechnical reasons that might be triggered by seepage or piping.

(3) Provides an early warning system, allowing starting emergency measures when unfavorable developments are detected.

19.2 River Monitoring

Monitoring of river provides the data on morphological behavior of the river that facilitates planning active interventions. Large-scale bathymetric surveys, undertaken at the end of the monsoon season, verify the morphological prediction of river changes and provide baseline data for the work during the dry season. Emergency repair work during monsoon is also facilitated by the regular monitoring and surveys. Important additional information collected is (i) maximum river depth, (ii) flow velocities, and (iii) angles of attack on protective works (through float tracking and ADCP measurements).

Bankline or structural surveys provide important data related to riverbank protection work. Data collected shall facilitate assessing (i) changes of the bankline, (ii) river conditions during construction phases and after the construction, (iii) riverbank slopes and (iv) development of falling aprons used as toe protection or launching heaps used as immediate protection in case of emergencies.

All post-construction data forms the baseline data for assessing the condition and behavior of a structure under different load. In addition these surveys allow assessing the risk for surpassing navigation limits or in case of dredging the risk of damages to the established work.

The importance and necessity for monitoring of river behavior as explained above clearly dictates the monitoring of following activities:

(1) Cross-section survey at an interval of 500m for major rivers and 200m in case of medium rivers during monsoon and immediately after the monsoon,

(2) The survey mentioned under (i) shall cover at least 5km at both upstream and downstream of a protection structure,

(3) In case of severe attack on a structure the cross-section at an interval of 100m or closer shall be carried out,

(4) Flow tracking and measurement of flow velocity shall be carried out before monsoon, during monsoon and after monsoon around all protection structure at least for a length of 5 km covering both upstream and downstream,

(5) Collection of satellite image for the area (each year) covering the protection structure and also at least 10 km upstream and downstream (influence area) to study the expected morphological changes.

19.3 Operation Monitoring

Continuous observation and monitoring is required to record the response of the river on bank protection works and also the morphological changes afterwards. Monitoring does not only refer to detecting damages on the structures but to observe their behavior under load and to relate the loads to the structure's response.

The purpose of monitoring and assessment activities is to ensure the standard of performance of bank protection work. Monitoring will mainly concentrate on areas where bank protection work is implemented. Operation monitoring includes :

(1) Morphological condition of the river,

(2) Performance of the structure during flow attack

Morphological condition may be monitored to observe

- any abnormal flow attack on the structure and
- through regular (once a weak in flood period and monthly or quarterly in lean flow season) bed level survey of the area to detect any abnormal scour, flow pattern, flow velocity etc.

Cross-sections at about 10 m interval extending to both upstream and downstream of protection and the deepest part of the river need to be repeated every year before and after the monsoon and a continuous record maintained. From the survey section a contour field at 0.5m intervals may be of more use.

Regular visual inspection of the structure is necessary prior to and immediately after the floods and strong winds to detect weak areas of the structures. In addition, daily inspection is necessary during floods and windy periods in inclement weather.

A continuous monitoring of the executed protection works is needed to assess the morphological behavior of the river in that area. Any particular protected reach may need strengthening or any reach may need to be extended on emergency basis as detected through monitoring.

19.4 River Survey

Survey works to monitor the performance of the protection work already implemented and also to observe overall morphology of the river is needed on a regular basis. In that case bathymetric survey of an appreciable area around the implemented structure is necessary. The survey data would be helpful to visualize and understand the response of the river on implemented protection and also the morphological behavior as a whole. Velocity data also helps analyzing the potential areas of flow attack as well a scour zone.

19.5 Maintenance

Maintenance should extend over the lifetime of the structure/protection work. Ideally there could be no maintenance free bank protection structure. But in reality in Bangladesh most of the structures are left to nature for their performance which shouldn't be the case. All structures require regular maintenance through proper monitoring.

The key objective of the maintenance is to keep the system in good operating condition with minimum cost of repair avoiding rehabilitation/reconstruction. Maintenance is primarily repair/reinforcement of weak or damaged part of a structure and also is part of the management of the hydraulic structure that mainly includes inspection and repair.

Experience with bank protection structures shows that the critical time is the first year immediately after construction, when the river fully tests the new structure. Up to 30% of the total investment cost may be required after the first flood to rebuild or strengthen the structure. The repair and remedial works on areas detected through survey record should follow in a systematic way and be undertaken within a short period.

In general along bankline of major river of Bangladesh a stable bank slope is in the order of IV: 2H. So to attain a sustainable bank protection works the steep underwater slopes (slope steeper than IV: 2H) may need to have special treatment. The steep sections can be made flatter to IV: 2H by filling with earth filled jute bags and jute bag slope can be covered with at least two layers of hard materials/geo-bags for a more or less permanent protection. This type of work demands accuracy in placing/dumping of protecting materials in proper position. The special treatment as suggested may not be required if the overall slope from top of bank line to bottom of the bank slope is 1V:2H.

Observation made in several cases show that the scour hole developed during monsoon is completely filled up during the receding flood. In such cases it is needed to take proper care to build the underwater slope during monsoon whenever a long stretch of slope is detected to be steeper than IV: 2H.

The provision of maintenance shall be kept on the basis of standard design, expected length and depth of river. The block allocation provided for such maintenance shall be finalized after the pre-work survey conducted at the end of monsoon. Abnormal behaviour of structure and any morphological change in its vicinity during monsoon shall also have to be considered. Any emergency repair needed in those reaches or any other vulnerable reach already identified through prediction shall be taken care from block allocation maintained from the O&M budget.

If the identification of any deterioration or damage is not done regularly and repair is not done in time, major damage may happen requiring reconstruction /rehabilitation involving huge cost in the long run.

References

REFERENCES

1. A.A.Balkema (1991). Manual on the use of rock in coastal and shoreline engineering, CIRIA special publication 83/CUR report 154
2. Ahmad, M. (1953). Experiments on Design and Behaviour of Spar dikes, Proc. of 5th Cong. of IAHR.
3. Alam, M.K, Fontijn, H.L. (2006). Design Manual for Placed-Block Revetments: Wave Protection in Haor Areas. Final Technical Report, BUET-DUT Linkage Project, Phase-III, BUET.
4. Annual Book of ASTM Standards, (1989). vol.04.08.
5. B.Przedwqjski, R.Blazejewski and K.W.Pilarczyk. River Training Techniques; Fundamentals, Design and Applications.
6. Bari, M.F. and Saleque, G. (1995). Flood flows of the Jamuna: A study of frequencies and magnitudes, Jour. Inst of Engrs, Bangladesh.
7. Benson, M.A. (1960). Characteristics of frequency curves based on tehotetical 1000 year record, USGS Water Supply Paper 1543-A.
8. Bhuiyan, A.B.M. F., Pankaj, M. and Doescher, T. (2006). Morphodynamics around bank protection structures in the large braided river Jamuna. Proc. 15th APD-IAHR Congress, Chennai, India, Vol I, 311-320.
9. Bhuiyan, A.B.M.F., Hossain, M.M. and Hey, R. D. (2001). Bank erosion and protection on the Barhmaputra (Jamuna) River. Hydraulic Information Management, Section 4, Eds. W.R. Blain and C.R. Brebbia, WIT press, Southapton, Boston, pages: 173-182.
10. Bhuiyan. A.B.M.F.(2007). River Erosion Mitigation and Sediment Control Technology in Bangladesh, National Workshop on Soil, Erosion, Eco-Degradation and poverty Relationship in Bangladesh perspective, (IECABC).
11. Boeters, R. E. A. M., Verheij, H. J., and Wal, M. v. d. (1991). Environment-friendly bank protection. Int. Sym. on Environmental Hydraulics, Hong Kong, 1-6.
12. Breusers, H.N.C and Raudkivi, A. J. (1991). Scouring, Hydraulic Structures Design Manual, IAHR.
13. BRTC, BUET (2001). Technical evaluation of some selected flood protection and river erosion control works already implemented by BWDB. Final Report, Department of Water Resources Engineering, Bureau of Research Testing and Consultancy, BUET.
14. BRTC, BUET, (1998). Investigation into the damage to the works of contract B2 Sirajganj during monsoon. Report, Bureau of Research Testing and Consultancy, BUET.
15. BRTC, BUET, (1998). River Bank Protection Project, Brahmaputra Right Bank Priority works, Investigation into the damage to the works of contract B2 at Sirajganj During Monsoon.
16. BRTC, BUET, (2002). Technical Evaluation of Some Selected Flood Protection and River Erosion Control Works Already Implemented by BWDB.
17. BWDB(2004). Report of the Committee on Assessment & Evaluation of Protective Works Since Independence and Reviewing of Existing Sand Extraction and Dredging Program, Bangladesh Water Development Board, Ministry of Water Resources, Government of Bangladesh.
18. BWDB, (2002). Tech. evaluation of some selected, Flood protection and River Erosion Control Works already Implemented by BWDB. Draft Final Report, BWDB.
19. BWDB, (2004).Assessment & Evaluation of Protective works since Independence and Reviewing of Existing Sand Extraction and Dredging Program.

20. BWDB, (2005). A Concise Technical Report on Low Cost Bank Protection Works in Different Rivers of Bangladesh, Chief Engineer, Design, Bangladesh Water Development Board, Dhaka.
21. BWDB, (993). Guide to Planning and Design of River Training and Bank Protection Works; Design Circle-II, BWDB, Dhaka.
22. CEGIS(2005). Prediction for Bank Erosion and Morphological Changes of the Jamuna and Padma Rivers 2005; July 2005; (Prepared by CEGIS for EMIN Project of WARPO and JMREM Project of BWDB).
23. CEGIS, (2004). Monitoring and Prediction of Bank Erosion along the Right Bank of Jamuna River.
24. CEGIS, (2004). Morphological Analyses and Predictions for the Jamuna River around FAP 21 test structures.
25. CEGIS, (2004). Morphological Analyses and Predictions for the Jamuna River around the PIRDP site 2004; Prepared for the Jamuna Meghna River Erosion Mitigation Project, BWDB.
26. CEGIS, (2004). Morphological Predictions for the Padma and Meghna Rivers at MDIP site and Chandpur, Prepared for the Jamuna Meghna River Erosion Mitigation Project, BWDB.
27. CEGIS, (2005). Developing Empirical Methods for Predicting Morphological Changes in the Padma River; Prepared for Jamuna Meghna River Erosion Mitigation Project.
28. Chang, H. H (1992), Fluvial Process in River Engineering, Krieger Publishing Company. Florida.
29. Chow, V.T. (1951). A general formula for hydrologic frequency analysis, Trans. Am. Geophysical Union, vol. 32, no. 2, pp. 231-237
30. Chow, V.T. (1959). Open Channel Hydraulics, McGraw Hill Book Company
31. Chow, V.T., Maidment, D.R., Mays, L.W. (1988). Applied Hydrology, McGraw-Hill Book Co.
32. Command Area Development Project (CADP), (2000). River Erosion Prevention and Morphology Study; PIRDP; Annex-B: River Training Final Report.
33. Cunnane, C. (1978). Unbiased plotting positions – a review, J. Hydrol, vol. 37, pp. 205-222.
34. CUR/CIRIA, (1991). Manual on use of rock in coastal engineering, CUR/CIRIA Report 154 Gouda, Netherlands.
35. Earthquake Engineering Handbook, Wai-Fah Chen, CRC Press, 2003.
36. FAP 16 and FAP 19, (1995). The Dynamic Physical and Human Environment of Riverine Charlands: Brahmaputra-Jamuna, ISPAN.
37. FAP 21, (2001). Guidelines and Design Manual for Standardized Bank Protection Structures; Bank Protection Pilot Project; Water Resources Planning Organization, Ministry of Water Resources; Government of Bangladesh.
38. FAP 21/22 (1993). Final Report, Planning Study; Volume 1A; Main Report on Bank Protection; FAP 21; Bank Protection and River Training (AFPM); Pilot Project.
39. FAP 21/22 (2001). Final Project Evaluation Report; Volume I; Main Report; Bank Protection and River Training (AFPM); Pilot Project; FAP 21/22; December 2001; Consulting Consortium FAP 21/22, RHEIN-RUHR ING. GES MBH, Dortmund, Germany.
40. FAP 21/22, (1993). Main Report on Bank Protection; FAP 21; Volume 1A: Bank Protection and River Training (AFPM); Pilot Project; FAP 21/22; Final Report, Planning Study; Consulting Consortium, RHEIN-RUHR ING. GES.MBH, Dortmund, Germany.

41. FAP 24, (1996). River Survey Project, Main Volume, Final Report.
42. FAP-24, (1996). River Survey Project, Main Volume, Final Report.
43. Gales, R. (1938). The Principles of River Training for Railway Bridges and their Application to the case of the Hardinge Bridge over the Lower Ganges of Sara. Journal of the Institution of Civil Engineers, December 1938. Paper No, 5167, India
44. German Federal Waterway Engineering and Research Institute (2005). Principles ffor the design of bank and bottom protection for inland waterways. Bulletin No. 88.
45. Haan, C.T. (1977). Statistical Methods in Hydrology, Iowa university Press, Ames, Iowa.
46. Hacrow S.W. & Partners Ltd. (1998). Review of River Bank Protection Project, Engineer's Review and Action Plan Report on Damages during Monsoon.
47. Halcrow and EPC (1998). Engineers review and action plan. Report on damages in 1998 monsoon, Contract B2-Sirajganj.
48. Halcrow in association with Haskoning, BWDB, River Bank Protection Project. October 1999.
49. Hasselmann, K. et al (1973). Measurement of Wind-Wave Growth and Swell Decay during the Joint North Sea Wave Project (JONSWP), Deutsches Hydrographisches Institut, Humburg.
50. Hemphill, R. W., and Bramley, M. E. (1989). Protection of River and Canal Banks - a guide to selection and design - CIRIA Water Engineering Report, Butterworths, London.
51. Hoque, M.M. Ahmed, S.M.U. and Siddique, A.B. (1988). Generation of boundary hydrographs for flood prediction in the major rivers of Bangladesh, Bangladesh Floods, Views from home and abroad, University Press Ltd., Dhaka.
52. Hossain et al (2007). Morphological characteristics of Arial Khan river in the vicinity of Arial Khan Bridge in Bangladesh, Proceedings of the International Conference on Water and Flood Management (ICWFM 2007), Dhaka, Bangladesh, Vol. I, pp. 241-248.
53. Hossain, M. M. (1989). Geomorphic Characteristics of the River Ganges upto Brahmaputra (Jamuna) Confluence. Research Project conducted at IFCDR, BUET, as Principal Investigator.
54. Hossain, M.M. (1992). Total sediment load in the lower Ganges and Jamuna, Journal of the IEB, Vol. 20, No. 1&2, pp. 1-8.
55. Inglish, C.C. (1949). The effect of of variations in charge and grade on the slopes and shapes of channels. In Proc. 3rd Congr. IAHR, Grenable, Vol. 1, Paper II, pp. II.1.1-II.1.10.
56. International Society of Soil Mechanics and Geotechnical Engineering, Technical Committee TC-33 on Scour of Foundations, (2000). Scour of Foundations; Proceedings of an International Symposium held in Melbourne, Australia.
57. JMREMP (2005). Special Report 10: River Morphologist Mission Report; May 2005; Jamuna Meghna River Erosion Mitigation Project: Part-B.
58. JMREMP (2005). Special Report 11: Physical Model Study (Vancouver, Canada); May 2005; Jamuna Meghna River Erosion Mitigation Project: Part-B.
59. JMREMP (2006). Special Report 23: Design Brief for River Bank Protewction implemented under JMREMP. Dec 2006; Jamuna Meghna River Erosion Mitigation Project: Part-B.
60. JMREMP(2002). Feasibility Study- Final Report: Addendum, October 2002, Jamuna Meghna River Erosion Mitigation Project (Phase-II),
61. JMREMP(2002). Feasibility Study- Final Report: Volume 1. 2 & 3; May 2002; Jamuna Meghna River Erosion Mitigation Project (Phase-II),
62. JMREMP(2003). Geo-bag Protection: Project Concept, Experience, Future (2003). Jamuna Meghna River Erosion Mitigation Project- Part-A.

63. JMREMP(2003). Quarterly Report, April- June 2003; Jamuna-Meghna River Erosion Mitigation Project, Part-A.
64. JMREMP(2004). Draft Implementation Plan; Dry Season Work 2003/2004; Jamuna-Meghna River Erosion Mitigation Project; Part A.
65. JMREMP(2005). Construction Completion Report. Contract Package No.Pl, P2 and Ml, Jamuna Meghna river Erosion Mitigation Project; Part-B.
66. JMREMP(2005). Inception Report, (2005). Jamuna Meghna River Erosion Mitigation Project: Part-B.
67. JMREMP(2005). Inception Report, January 2005; Jamuna Meghna River Erosion Mitigation Project: Part-B.
68. JMREMP(2005). Operation and Maintenance Manual (Draft updated annually); July 2005; Jamuna Meghna River Erosion Mitigation Project: Part-B.
69. Joglekar, D.V (1971). River Behaviour, Control and Training.
70. Kendall, M.G., and Stuart, A. (1967). The Advanced Theory of Statistics, vol. 2, 2nd ed., Hafner, New York
71. Kite, G.W. (1988). Frequency and Risk Analysis in Hydrology, Water Resources Publications, Fort Collins, Colorado
72. Klaassen, G.J. and Vermeer, K. (1988). Confluence scour in large braided rivers with fine bed material. Proc. of International Conference on Fluvial Hydraulics, Budapest, PP. 395-408, (after FAP 21, 2001)
73. Linsley, R.K. Jr., Kohler, M.A., and Paulhus, J.L.H. (1988). Hydrology for Engineers, McGraw-Hill Book Co.
74. Margaret S. Petersn (1986). Rivr Engineering, Prentice-Hall.
75. Neil C. R (1973) Guide to bridge hydraulics, Toronto and Baffalo, RTAC, Univ. of Toronto press.
76. Neil C. R (1980) Metric Version of Guide to bridge hydraulics, Toronto and Baffalo, RTAC, Univ. of Toronto press.
77. NERC, (1975). Flood studies report, Vol 1, Hydrological studies, Natural Envionment Research Council, London, UK.
78. Northwest Hydraulic Consultant, NHC, (2006). JMREMP, Physical model study, final report, Vancouver, Canada.
79. NWMP, (1999). Hacrow S.W. & Partners Ltd and Mott MacDonald, Review of River Bank Protection work, NWMP.
80. Oberhagemann, K. and Noor.A. P.Eng. (2004). Development of Bank Protection Solutions in Bangladesh, 1th International Conference on Managing Rivers in the 21st century: Issues & Challenges Rivers, pp. 208-214.
81. Oberhagemann, K. and Sharif-Al-Kamal M. (2004). Geobag Protection in Bangladesh, 4th International Conference on Filters and Drainage in Geotechnical and Environmental Engineering Geofilters, pp. 205-215.
82. Odgaard, A. J., and Kennedy, J. F. (1983). River-bend bank protection by submerged vanes. J. Hydr. Engrg. ASCE, 109(8), 1161-1173.
83. Odgaard, A. J., and Spoljaric, A. (1986). Sediment control by submerged vanes. J. Hydr. Engrg. ASCE, 112(12), 1164.
84. Office of Technology Applications, HTA-22 Federal Highway Administration Washington D.C, Evaluating Scour at Bridges, Third Edition, Hydraulic Engineering Circular No. 18, Publication No. FHWA HI-96-031, Nov. 1995.
85. Office of Technology Applications, HTA-22 Federal Highway Administration Washington D.C, and Stream Stability at Highway structures, Second Edition, Hydraulic Engineering Circular No. 20, and Publication No. FHWA HI-96-032, Nov. 1995.

86. PIANC, (1987). Guidelines for the Design and Construction of Flexible Revetments incorporating Geotextiles for Inland Waterways, Report of Working group 4, Supplement to Bulletin No.57, Belgium.
87. Pilarczyk, K.W (1998). Dikes and Revetments, Design, maintenance and Safety Assessment, Balkema.
88. Pilarczyk, K.W. (1990). Coastal Protectio, A.A.Balkema (publisher), Rotterdam.
89. Pilarczyk, K.W. (2008). Design of Alternative revetments. Handbook of Coastal and Ocean Engineering, edited by Kim Young, World Scientific publishers..
90. Przedwojski, B, Blazejewski, B and, Pilarczyk, K.W (1995), River Training Techniquies: Fundamentals, Design and Applications, Balkema.
91. Remillieux, M. (1967). Reducing the investment of river regulation with the new method of bottom panels. Int. Conf. Water and Peace, Washington D. C.
92. Report of Working Group of the Permanent technical Committee1, (1987). Guidelines for the Design and Construction of Flexible Revetments Incorrupting Geotextiles for Inland Waterways, Permanent International Association of Navigation Congresses, Supplement to Bulletin N0 57.
93. Richardson, E.V, Stevens, M.A, Simons, D.B, (1975). The design of Spurs for River Training, xvth congress of IAHR, Sao Paulo, Brazil;, Vol. 2, paper 347, pp. 382-388.
94. RSP(1996). Final Report, Main Volume, River Survey Project, FAP 24.
95. Saville, T., JR. (1952). Wind Setup and Waves in Shallow Water, TM-27, U.S Army Corps of Engineers, Beach Erosion Board, Washinmgton D.C.
96. Sping, F.J. (1903). River Training and Control on the Guide Bank System, Railway Board Technical Paper No. 153, New Delhi.
97. Standard Design Manual; Chief Engineer, Design, Bangladesh Water Development Board, Dhaka.
98. Stanley A. Schumm and Brien R.Winkley, (1994). Variability of Large Alluvial Rivers;: ASCE Press, New York.
99. Stevens, M.A. and Oberhagemann, K. (2006). Geobag Revetments, Special Report 17; JMREMP.
100. Thorne, C.R (1988). Bank Processes on the Red River between Index, Arkansas and Shreveport, Louisiana, Final Report to U.S Army ERO, Dept. of Geography, Queen Mary College, London.
101. U.S Army Corps of Engineers (1991). Hydraulic Design of Flood Control Channels, Engineering and Design, EM 1110-2-1601.
102. Uddin. M.J., RCC Spur in Bangladesh Review of Design, Construction and Performance, M.Sc. Thesis WSE-HERBD-07-17, UNESCO-IHE, April 2007.
103. United States Army and River Dynamics Group, European Research Office London, England, (1995). Guidebook of Applied Fluvial Geomorphology for River Engineering and Management.
104. US Department of Transportation, Federal Highway Administration (2001). Bridge Scour and Stream Instability Countermeasures, Experience, Selection, and Design Guidance Second Edition, Hydraulic Engineering Circular No. 23, and Publication No. FHWA NHI-01-003.
105. Varma, C.V.J., Saxena, K.R. and Rao, M.K. (1989). River Behaviour Mangement and Training. Pub No 204, Vol. 1, Central Board of Irrigation and Power, New Delhi.
106. Viessman, W. Jr. and Lewis, G.L., (1996). Introduction to Hydrology, Harper Collins College Publishers, 4th ed.
107. WARPO, Water Resources Planning Organization (2001). National Water Management Plan., Ministry of Water Resources, Bangladesh.

108. Water Resources Council (now called Interagency Advisory Committee on Water Data) (1981). Guidelines for determining flood flow frequency, bulletin 17B, US Geological Survey, Reston, VA 22092

Appendix

APPENDIX - A

Experience of Selected Bank Protection Works on Major Rivers in Bangladesh

A.1 Introduction

Experiences and salient features of selected bank protection works, mainly revetments and groynes, as observed from the records and visits by the consultant team are presented here. Some background information of the structure, important design considerations, damage records and relevant costs where available are briefly explained. General remarks are given at the end of the chapter.

A.2 Implemented Bank Protection Works

Bank protection works on the major rivers were mainly undertaken on the basis of need, e.g. Hardinge Bridge and Rajshahi Town protection works on the Ganges and Sirajganj Town protection on the Jamuna. Chandpur Town protection works on the river Meghna is also an example of earlier bank protection works.

Various types of bank protection and river trainings works were executed in Bangladesh in the past. These are revetments, groynes or spurs, porcupines, cut-off, retaining walls etc. Among these revetments and groynes are more or less permanent measures, which have been used widely in the recent time.

About 416 km bank revetment and 207 groynes or spurs were constructed by Bangladesh Water Development Board (BWDB) all over Bangladesh at a cost of about BDT 3,032 crore of which 366 km revetment and 195 spurs (88% and 95% of total, respectively) are still working satisfactorily (MoWR 2004). In all the protective works, the hard materials used are natural boulder, quarried stone, boulder or stone crate, concrete cube, concrete slab (slope protection), sand-cement block, sand-cement bag and geotextile bag filled with sand. In some of the temporary and emergency protection works jute bags or synthetic bags filled with sand (earth), brick crates and geotextile bags filled with sand have been used. Fig. A.1 shows locations of major protection works along the Brahmaputra-Jamuna River.

Low-cost temporary measures like porcupines (bamboo crates filled with bricks) placed at the toe of banks and lower parts of the slopes have been found to be effective only in smaller silt-laden rivers. However, its use in flashy rivers like the Teesta (e.g. at Belka) or the Jamuna (e.g. at Fulchari Ghat) did not prove successful for stopping bank erosion.

A.3 Revetment works

A.3.1 Jamuna Bridge Guide Bund

Background

The Jamuna Multi Purpose Bridge (JMB) is situated about 8 km downstream of Sirajganj Town. Mainly to optimize the cost of constructing the bridge, the original river which was about 10 km wide had been reduced to 4.8 km by two guide bunds. The bridge was constructed during the period 1994-1998. The river engineering aspects concerned the stability of the channel configuration and the width of the braid belt by fixing the riverbanks in the vicinity of the

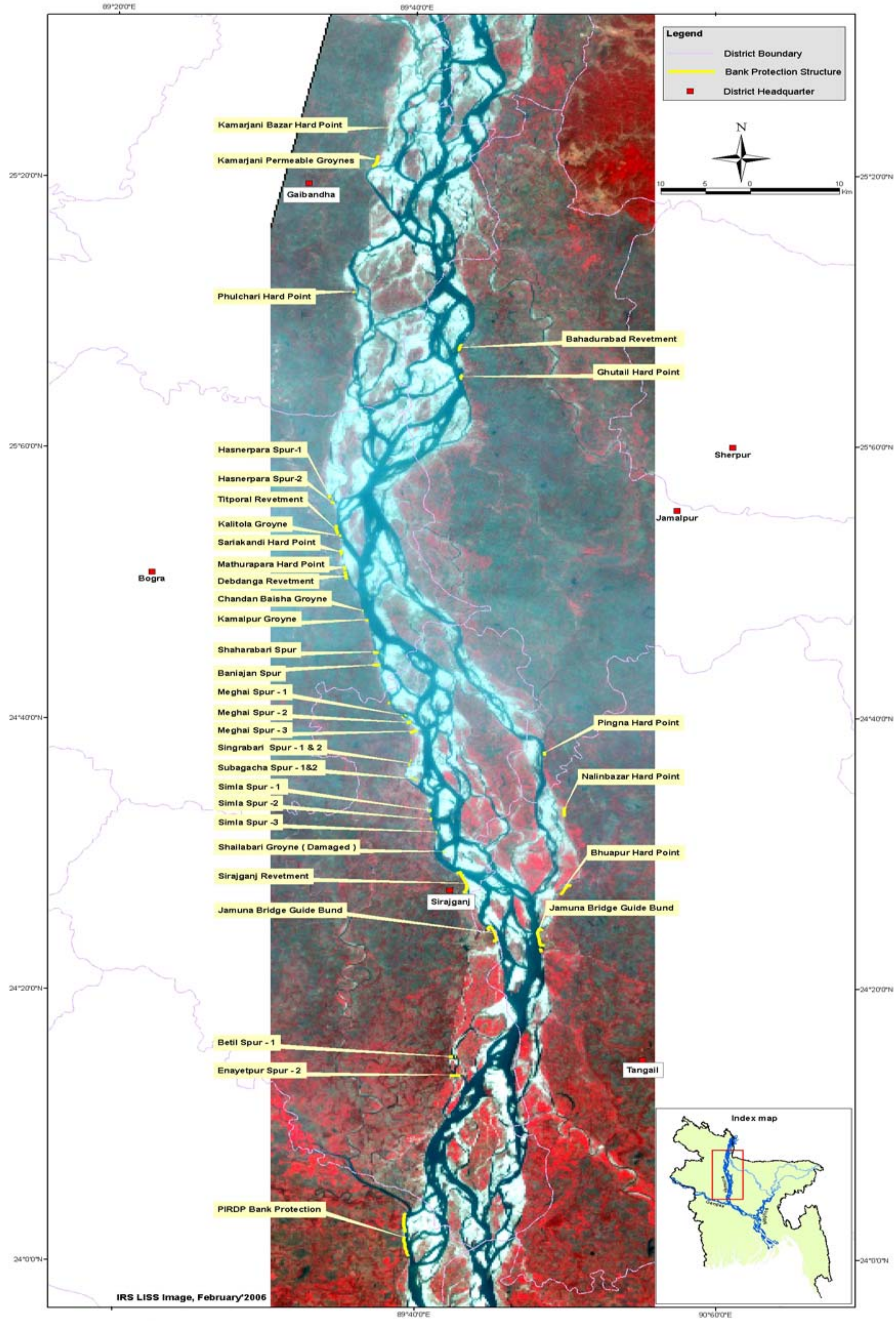


Fig. A.1 Bank protection structures along the Jamuna River (after CEGIS).

structure and guiding the flow through the bridge, which required a cost more than the original super structure.

Design and implementation aspects

The two guide bunds (Fig. A.4) each measures 3.2 km in length. They are heavily secured by slope protection work constructed on dredged slopes under still-water conditions. Apart from specially prepared geotextile (1 million m^2) and asphalt revetments, 1.5 million tons of rocks were placed on the slopes and in the falling apron. The supply of rock and boulders for the project was a major logistical operation itself. About 770 thousand tons of boulders were brought from rivers in the north-east of Bangladesh and 1.5 million tons of rock from quarries in India, Bhutan and Indonesia.

The falling apron was placed at a level of approximately -10.0 m PWD to -15.0 m PWD. The material used was graded quarried rocks with a size of 1-100 kg (D_{max}=0.3m). The quantity of rock per linear meter to allow scour down to a level of PWD -27 m is 34.9 m^3 per linear meter of guide bank. The rock volume in the falling apron is based on 5 layers, i.e. $5D_{50}$. The occurrence of a scour level of PWD -28 m was thought to have a 5% probability of occurrence.

The design considered 40 m to 45 m maximum scour depth. The design depth-averaged velocity was 2.1 to 4.3 m/sec. The mean annual sediment transport was 600 million tons with an average grain size (D_{50}) of 0.18 mm. No natural sufficiently stable sand layers could be expected in the top layers, which are usually prone to liquefaction and flow slides. Consequently, a gentle slope of 1V:5H had been chosen, as deep compaction turned out to be impossible.

Damages and maintenance works

Flow slides occurred during construction of the West Guide Bund (WGB) which led to reduced underwater slopes. Except this, no damage was reported till May 2006. Thereafter severe scour occurred at the WGB during the 2006 monsoon followed by first slides at the toe during .the 2007 monsoon (figure A-2 and figure A-3). A continuous well-organized monitoring system has been established for the structure. So any minor damage of the underwater works is repaired immediately.

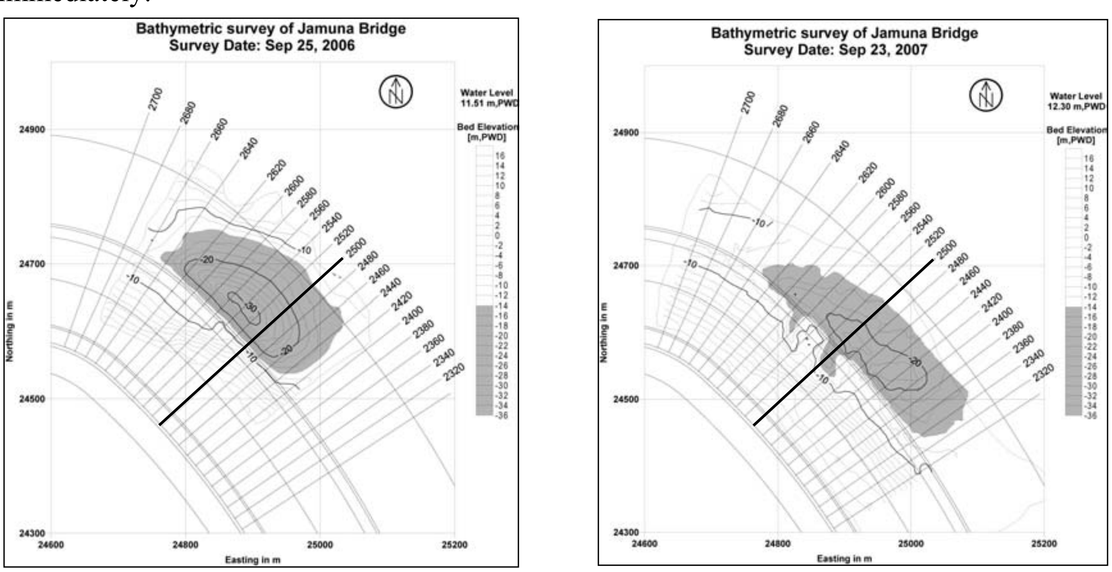
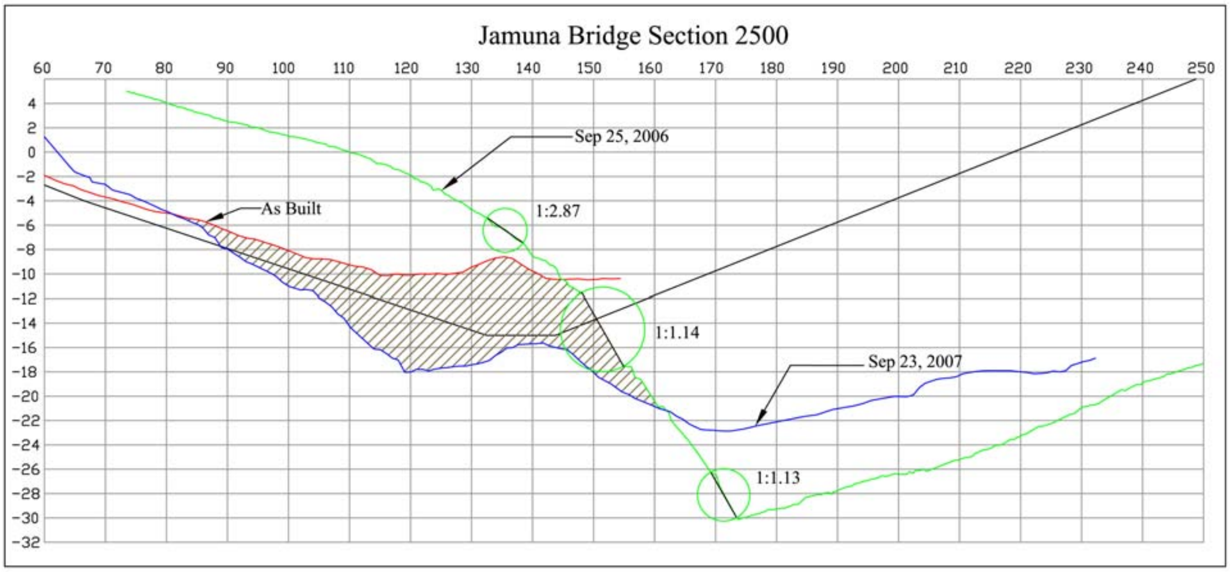


Figure A-2: Scour development at Jamuna Bridge WGB.



FigureA-3: Scour in Jamuna Bridge WGB

Cost

Total cost for construction of two, 3.2 km long each guide bund was, US$ 340 million. The construction cost of revetment was US$ 53,100 per meter (BDT 2,268,000/m, 1US$ = BDT 42.70), which is the most expensive protection work of this type ever built in Bangladesh.

Remarks

Due to extremely high cost involved, structure of this quality is not suitable for general bank protection purpose.

A.3.2 Bhuapur Hard Point

Works on the Jamuna Bridge include Bhuapur Hard Point in the left bank of Jamuna River, about 5 km upstream of the bridge (Figure. A-5).

Bhuapur Hard Point is designed to protect the banklines up to -20 m PWD. The structure consists of geotextile mattress, built in the dry land and is designed to develop with bank erosion.

The total cost involved was US$ 10.50 million for a 1550 m long revetment works. Thus the cost per km was about US$ 6.80 million or US$ 6,800 per m (BDT 289,000/m; 1US$ = BDT 42.70)

The revetment has not yet subjected to any river attack, so no experience is available about the long term stability.

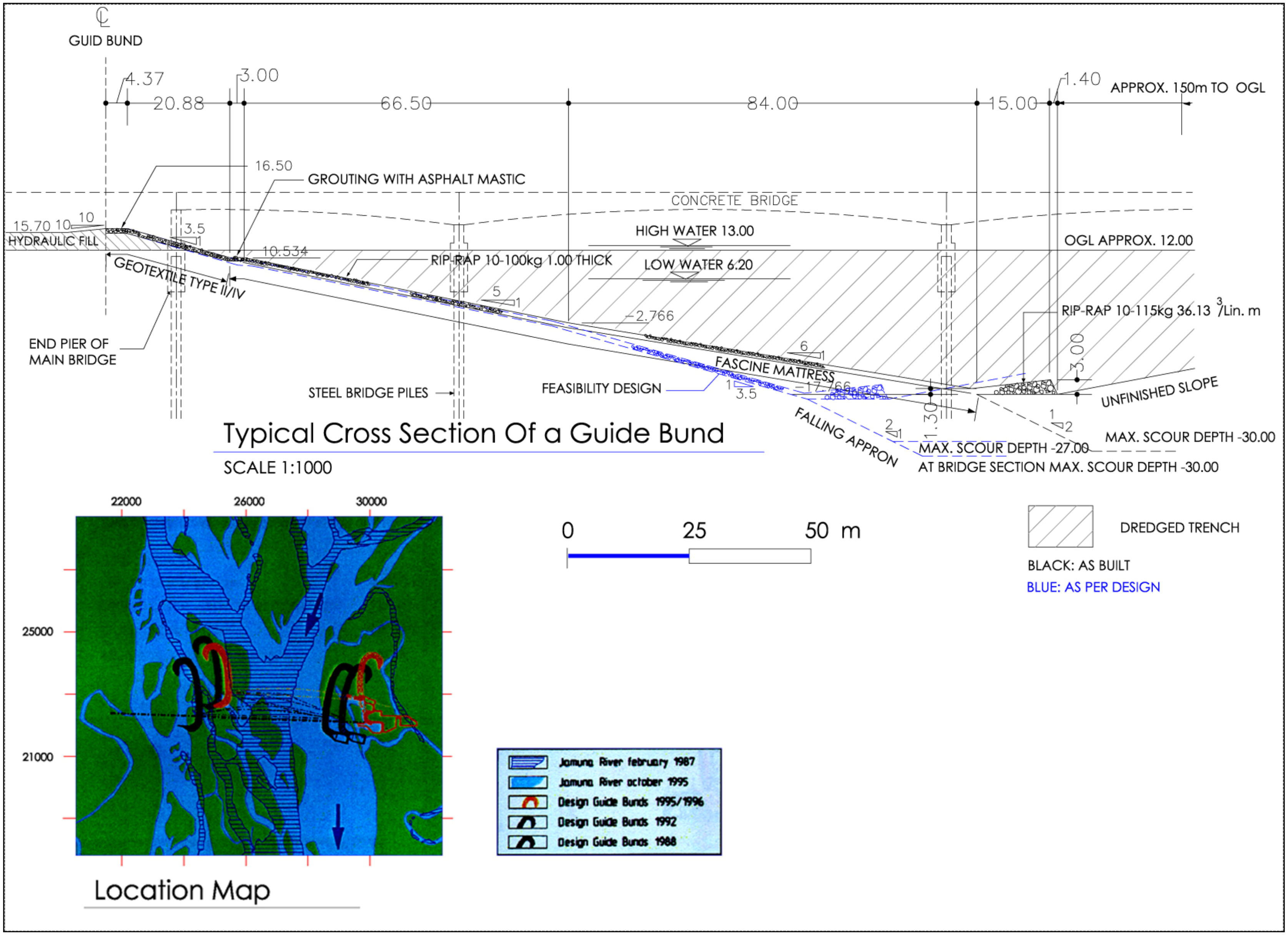


Figure A-4: Typical cross-section of Jamuna Bridge guide bund.

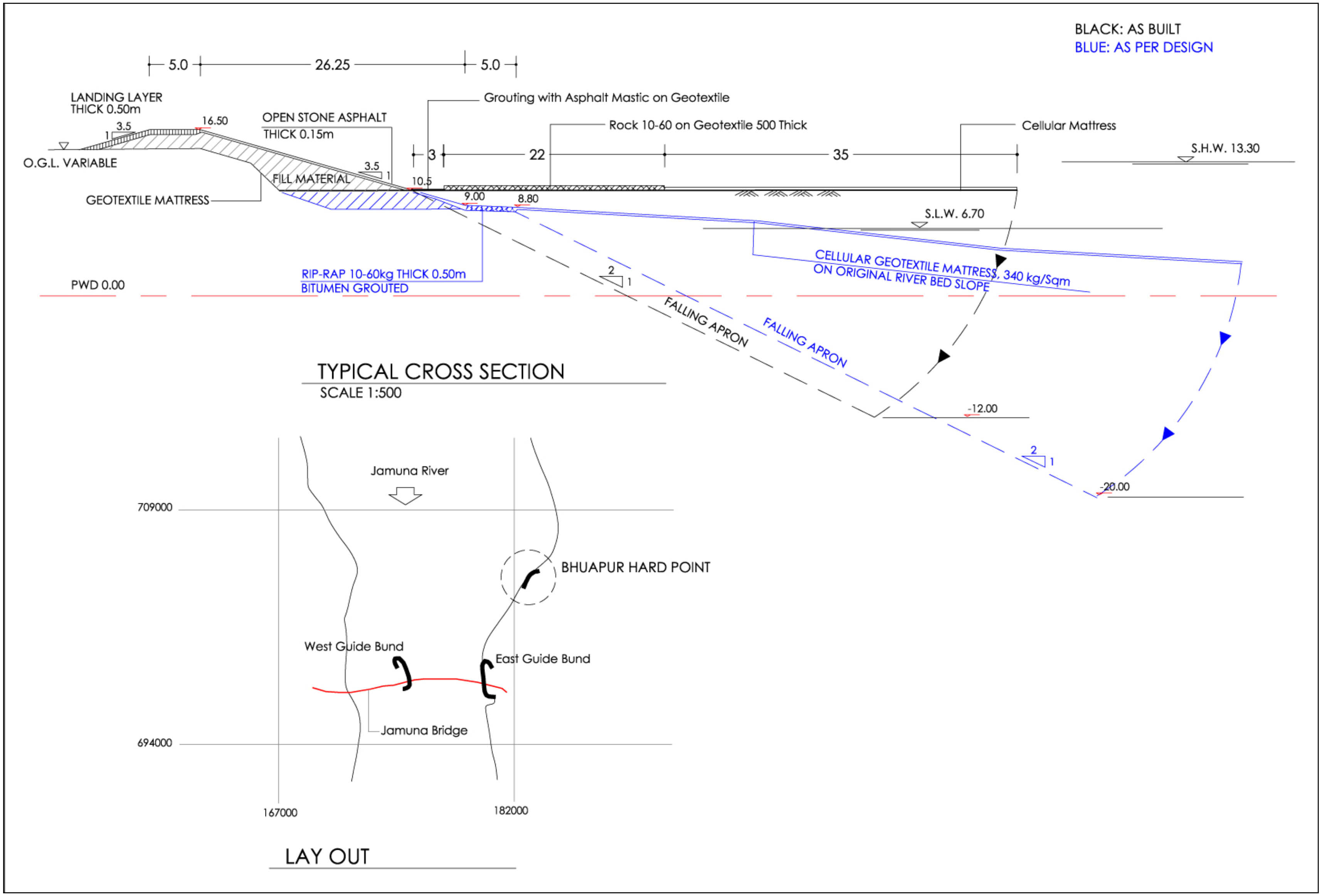


Figure A-5: Typical cross-section and layout of Bhuapur hard point..

A.3.3 Sirajganj Hard point

Background

Sirajganj town protection work started in the British period. The work was strengthened in 1964 with brick mattressing, which was washed away in 1969. During the seventies more than 2 km revetment made of sand-cement blocks was built but was again destroyed during 1985 flood. At that time the protection was enhanced with the construction of the Ranigram Groyne at slightly upstream of the town on the right bank of the Jamuna river. The groyne was 854 m long (305 m in water) and involved 62,300 m^3 of cc blocks. Army engineers were involved with BWDB personnel during the construction period. The construction was delayed due to fund problem and shortage of cc blocks. To cope with the situation, abandoned railway wagons had to be dumped at some places. This attempt was not, however, successful. In May 1986, a larger damage of the structure occurred whose value was estimated at about BDT 1 crore. Both the groyne and the revetment, which were provided with concrete block armour but no filter layer, were subjected to regular damages during monsoon.

To increase safety factor against anticipated severe erosion hazard, the Sirajganj Hard Point was reconstructed integrating the groyne and the revetment during the period 1996-1998 under River Bank Protection Project (RBPP) (Figure. A-6). Sirajganj Hard Point is important not only for the protection of the town but also a vital element to counter the widening of Jamuna River and to protect Jamuna Bridge in containing the river width within a fixed corridor at upstream locations.

Design and construction aspects

For design, the maximum wave height was estimated for the maximum fetch with an average water depth of 5 m. This resulted 1 meter wave height with 3 sec. period. The low water level and water level for 100 year flood following statistical analysis were +6.8 m PWD and +15.75 m PWD respectively.

Flow velocities were computed in several steps and final velocities were derived according to the following Table A.1.

Table A.1: Estimated velocities for different exceedance probability (m/s)

Parameter	Exceedance Probability [%]					
	2	5	8	10	20	50
u_{mean}	2.3	2.2	2.1	2.0	1.8	1.5
$u_{max\ no\ structure}$	4.0	3.6	3.3	3.2	2.8	2.2
U_{spur} (f=1.4)	5.7	5.1	4.8	4.6	4.0	3.0
$U_{revetment}$ (f=1.1)	4.5	4.0	3.7	3.6	3.1	2.3
$U_{revetment}$ (f=1.2)	4.8	4.3	4.1	3.9	3.4	2.6
$U_{revetment}$ (f=1.3)	5.2	4.7	4.4	4.2	3.7	2.8

f is the amplification factor due to structure determined from physical model test

Design flow velocities and resulting concrete block dimensions (khoa aggregates) are given in the Table A.2:

Table A.2: Design velocity and block size

Type/Location	Amplification factor [-]	Design velocity [m/s]	Calculated block size [m]	Selected block size [m]
Revetment, straight section	1.1	3.7	0.51	0.55 (0.45)
Revetment, upstream termination	1.3	4.4	0.71	0.72 (0.55)
Head of groyne	1.4	4.8	0.85	0.85 (0.65)

Note: figures in brackets are for stone aggregates.

The maximum water depth in the river was determined from statistical analysis of field observations and from regime equation with design discharge. The ratio of maximum depth to average depth was taken as 2.3 in the latter case. The maximum design water depth was obtained by adding local scour depth and dune height. Thus maximum water depth of 29 m for a 1:100 year flood event was estimated, while for the upstream termination 33 m of water depth was considered.

The falling apron was designed with two layers of cc cubes with an additional safety factor of 1.15. The apron was set at - 4.20 m PWD. The width of apron was determined to be 1.5 times the depth of scour hole. Thus the in-built reserve would be for 33 m deep scour for revetments and 44 m for the upstream termination and the spur head. The same slope of 1V:2H as for unprotected scour was applied following experiences from India. The design expectation was that, maximum scour up to -30 m PWD (45 m water depth) can occur but would be situated more than 100 m off the nose of the structure. Considering geotechnical stability the revetment slope of 1V:3.5H was found to be stable with a scour hole slope of 1V:2H which fulfilled the design safety factor.

A design lifetime of 30 years was selected for the concrete blocks being the most degradable element. One layer of cc-blocks was placed on the geotextile above low water level. Below water level, two layers (including 40% porosity) were placed for safety reasons. Uniform block dimensions were thought to be the best solution as the advantages of graded blocks were not proved. Khoa concrete with a specific weight of 1.98 t/m³ were considered at initial stage. Finally, stone aggregates (specific weight 2.24 t/m³) were used. The bank protection thickness was taken to be 850 mm, i.e., one layer of cc blocks plus 350 mm of brick filter placed on top of geotextile.

Construction of most of the structure in flowing water was considered. A design window of about 150 to 200 days (November – May) was estimated to be available. During this period the associated flow velocities at the site was found in the order of 0.7 to about 1.5 m/s.

Damages and Maintenance Works

Sirajganj hard point was severely damaged during the flood of 1998 (figure A-6). Scour observed in the vicinity of upstream nose was in the order of -33.0 m PWD. The corresponding depth of flow was 45.0 m PWD.

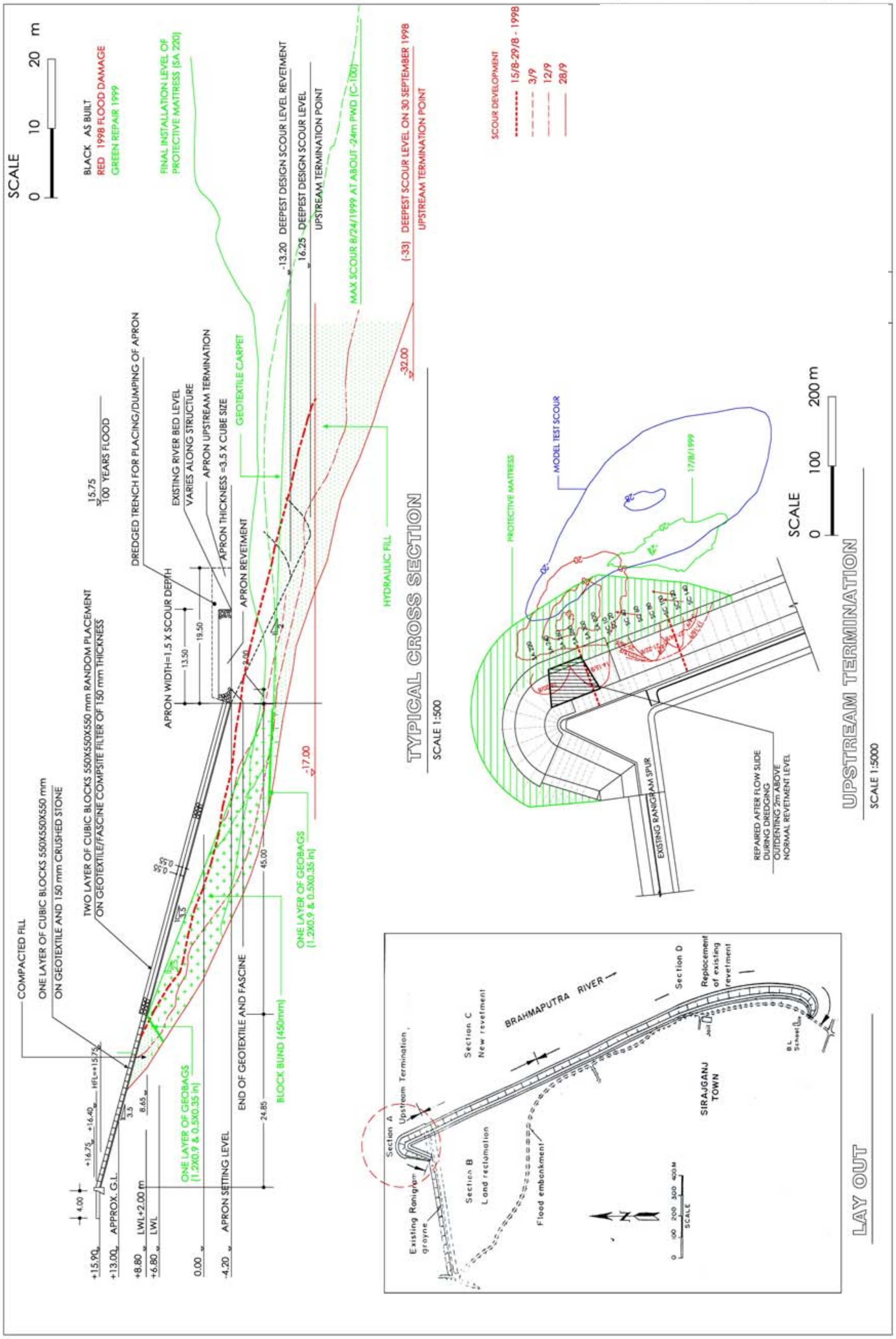


Fig. A-6 Typical cross-section and layout of Sirajganj hard point.

Main reasons for failure are the morphological changes creating unfavorable channel planform with deep confluence scour and the characteristics of subsoil (low relative density with mica). The sensitivity of the soil can be derived from the soil parameters such as degree of density of 32% with standard deviation 7% in the soil strata between +13 m PWD and –16 m PWD, resulting in low shear strength. The sensitivity of the slope to dredging was experienced during the establishment of the underwater slope when frequent slides occurred.

Repair works were executed during the dry season 1998-1999 by filling the eroded slope with a thick layer of cc blocks of sometimes more than 5 m thickness with a slope of 1V:2.5H. Geotextile blankets (mattresses of 15 x 5 x 0.1 m dimension weighing about 12 tons) were placed with total overlapping of 2.5 layers in the filled scour hole. Successive dumping of geotextile bags (pillows of 0.9 m x 0.75 m x 0.4 m weighing 250 kg and cushions of 0.4 m x 0.3 m x 0.15 m with 20 kg weight) were carried out to obtain a flexible scour carpet which reached up to 100 m from the toe into the river.

Experiences during the 1999 flood show that the dumped geotextile bags was successful in preventing from further deep scouring at the head of the structure. However, at the southern end of the carpet some flow slides happened due to undercutting and backscarp in the area where the flow separation line crossed the end of the carpet. In May, scouring started and maximum rates of 5 to 6 m/day was observed. Since beginning of June a total number of 18 flow slides occurred until end of August 1999.

The Sirajganj Hard Point at present is again under serious threat due to upstream embayment**.** Scour observed at the vicinity of upstream nose during the survey conducted in July-August, 2005 is in the order of -40 m PWD. This is known to be the deepest scour observed so far after the construction of hard points. The failure of the upstream Sailabari Groyne has contributed to the present condition of upstream embayment and resulting high scour near the upstream nose. Proper attention of arresting the westward migration of bankline upstream of the Sirajganj hard point is necessary to avoid such abnormal scour at the upstream area of hard point and risk of damages.

The extremely deep scour -40.00 m PWD occurred not due to a high flood but mainly for change of planform. So the river morphology is an important factor for designers and planners of bank protection works in rivers like Jamuna.

Cost

Prior to the RBPP work, total investment for Sirajganj Town Protection during the period 1972 to 1996 summed up about BDT 111 crore (1996 price). This includes 1988 flood damage rehabilitation cost, which is about 25% of the total amount.

For construction of the 2500 m long RBPP structure, US$ 73.5 million was required, which is about US$ 29,400 per meter (BDT 1,255,000 per meter; 1 US$ = BDT 42.70).

The repair cost of the 1998 damage was about US$ 15 million and proposition for repair after 1999 flood was US$ 1.9 million. These values were well above the estimated cost of maintenance.

Remarks

The damages of Sirajganj hard point have been analyzed by different group of experts. Some major lessons from the damage analyses are:

- Falling aprons sometimes do not work when rapid flow slides occur.
- Flatter revetment slopes, i.e. a more conservative design seems better. This leads to initially higher installation cost, but improves the lifetime of the structures significantly.
- Loose materials placed on the slope as ballast for the geotextile (cc-cubes in this case) has practically no resistance to flow slides.
- Thorough subsoil investigations are required.
- Hydraulic fill is problematic for the stability of underwater slopes and its properties need to be controlled during execution.
- Combination of unfavorable conditions due to morphodynamic processes that could have the same effect must be studied in detail.
- Maximum scour depth in the river seems to occur closer to protection structures compared to that observed in model studies.
- Over time repeated attack is observed for exposed structures like Sirajganj (figure A-7)

A.3.4 Sariakandi and Mathurapara Hard Points

Background

The Kalitola, Sariakandi and Mathurapara hard points were constructed to prevent Jamuna River from merging with the Bangali River that poses the risk of bypassing Jamuna Bridge. The three hard points constructed in a series were successful in stopping shifting of Jamuna River towards the Bangali River for a short period. The construction of the hard points was carried out during 1996-1998. Total length of the protection work was 4.5 km which protected about 6 km bankline of the river (Figure. A-8).

Design and construction aspects

Design floods for Sariakandi and Mathurapara hard points are same as described for Sirajganj Hard Point, only water level differs. Low water levels were +11.18 m PWD and +11.43 m PWD for Mathurapara and Sariakandi, respectively. Estimated water levels for the 1% flood event were +19.45 m PWD and +19.77 m PWD, respectively at the two locations.

Apron level for Sariakandi was at +0.43 m PWD and that for Mathurapara was at +0.18 m PWD. The revetment had a slope of 1V:3.5H from top to apron level.

Damages and maintenance works

After completion of construction, Mathurapara hard point experienced significant scouring (upto -7.3 m PWD) in the upstream point and the falling apron started to develop. In the early 1999, cc blocks were dumped to replenish the deployed apron close to the scour hole. During 1999 flood scour reached a level of -12 m PWD in the end of July. This was followed by siltation up to -7.7 m PWD in the end of August.

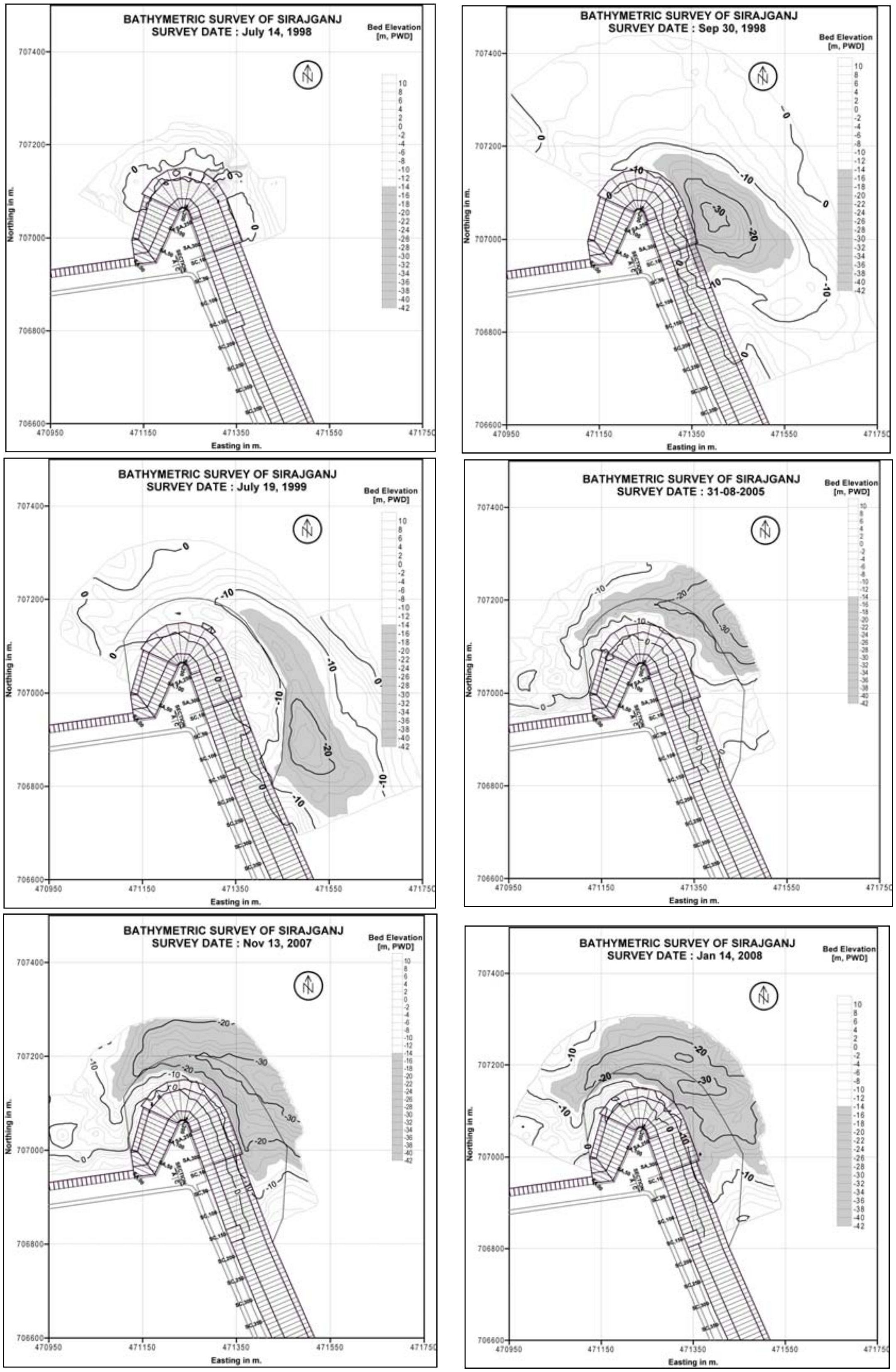


Figure A-7: Scour at Sirajganj Hard Point

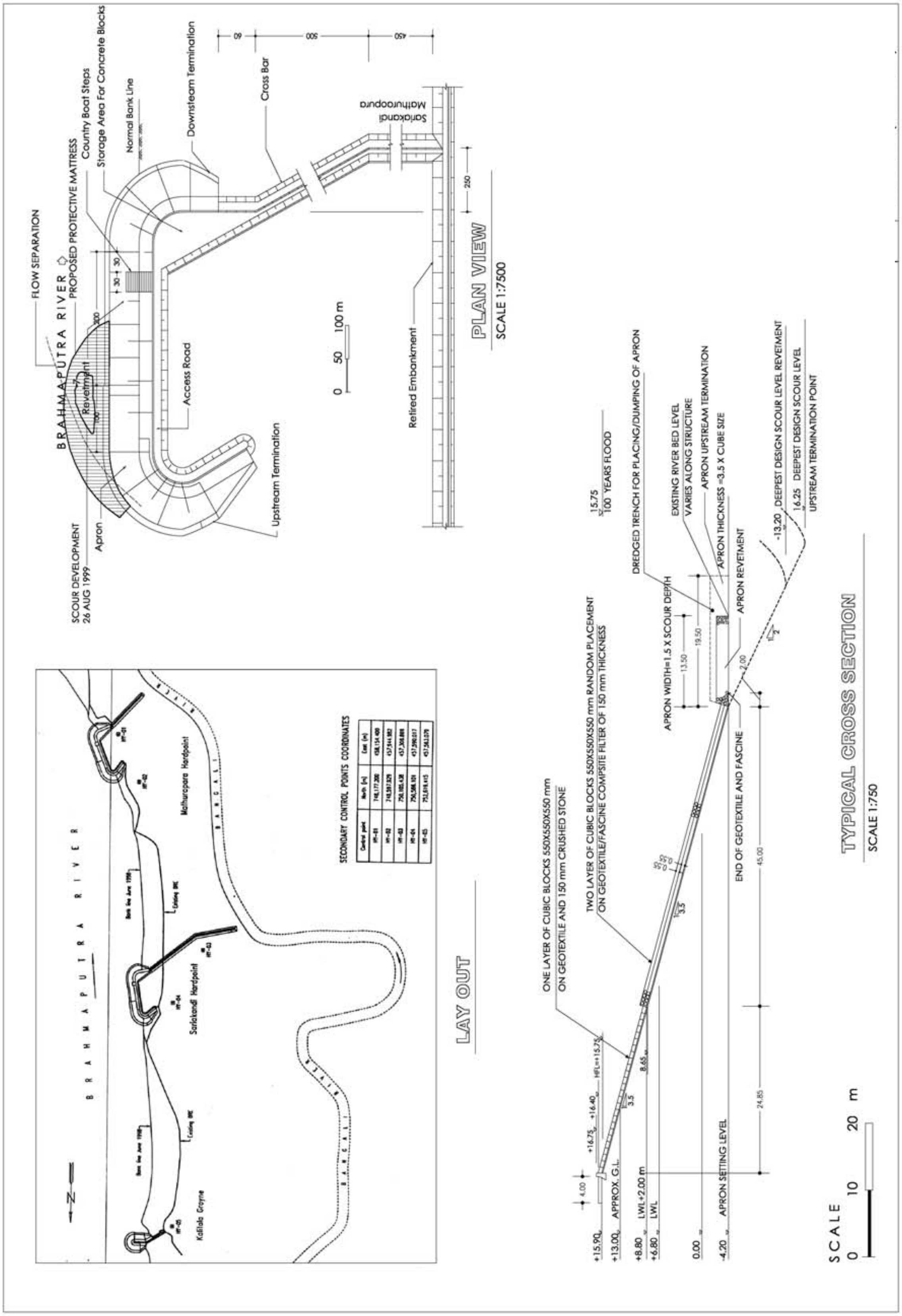


Fig. A-8 Layout and typical cross-section and layout of Sariakandi hard point.

Cost for construction of Sariakandi and Mathurapara Hard Point including refurbishment of Kalitola Groyne was US$ 52.1 million (BDT 222.50 crore, 1 US$ = BDT 42.70)

A.3.5 Revetment Works at Debdanga and Titporal

Background

Following the construction of the hard points at Kalitola, Sariakandi and Mathurapara, some morphological changes of the Jamuna has occurred so that the point of attack has been shifted. This has again created the possibility of merging the Jamuna with Bangali River in Titporol and Debdanga. These two sites are upstream and downstream of the above mentioned hard points, respectively. To avoid this situation and to prevent Kalitola and Mathurapara hard points from being outflanked, revetments have been constructed at Titporal and Debdanga along the right bank of Jamuna.

Design and construction aspects

The length of revetments at Titporol and Debdanga are 2000 m and 1273 m, respectively. The revetment works are constructed on a 1V:2H bank slope from 10 m PWD (LWL) to embankment crest level (about +16 m PWD) or floodplain level with one layer of cc block (400 x 400 x 200 mm) over a 3 mm thick non-woven needle punched geotextile filter. The toe protection (apron) has been achieved with 52 m^3 per meter of cc block placed on a 26 m wide apron from 10 m PWD to about 6 m PWD.

Immediately after construction of the revetment at Titporal and Debdanga, siltation on the apron has been observed. This demonstrates that some morphological changes have occurred in that reach of the river and the flow has been deflected from the zone of attack. However, during the flood 2007 erosion along the revetment occurred and partially damaged the revetment leading to additional maintenance requirements.

Figure A-9: Before and after the damage.

Cost

The cost of construction at Titporal is about BDT 61 crore i.e., BDT 305,000 per meter of the revetment works. The cost at Debdanga is BDT 42.00 crore (BDT 330,000 per meter).

Damagesand Maintenance works

After the construction no damages was observed in Debdanga portion of the revetment work. But in Titporal portion some failure in slopes were observed during 2007 and 2008. The estimated cost for the ongoing repair works in Titporal with 250 kg geobags and cc blocks is BDT. 68.60 million.

Remarks

After the construction of series of hard points at Kalitola, Sariakandi and Mathurapara in 1998, it has been observed that the river has changed point of attack with time, with the possibility of outflanking the protection work already constructed. Thus the problem for which the three hard points were constructed is still there but the location has shifted away. This indicates the necessity of an adaptive approach, applying continuous revetment with cheaper materials at first stage and then gradually making it stronger in phases as and when required.

A.3.6 Bahadurabad Test Revetment Structure

Background

Under the FAP 21/22 Bank Protection Pilot Project, the first revetment test structure was built at Bahadurabad in front of the village Kulkandi, situated at the east bank of the Jamuna about four hundred meter south of Bahadurabad railway ghat. One of the primary objectives of Bahadurabad revetment was the testing of the performance and stability of different revetment materials. The structure was built in 1996-1997.

Design and construction aspects

Terms of reference for Bahadurabad Revetment structure specified that low safety level had to be applied to study failure of the part of the structure to reach economic solutions without endangering the total protective work. The design concept reflected the testing purpose by establishing eight different sections with a total length of 800 m where different design alternatives and materials were tested (Figure. A-10).

During implementation, substantial contract modifications were made between contract and execution. The initial foreseen dredging solution was abandoned and replaced by a construction in the dry land (Figure. A-11). Even though the excavated and prepared ground was eroded at the beginning of 1996, the total cost was not changed.

Design wave height was considered to be 1 m with 3 sec period for 1% event. Water levels and flow velocities with a return period of 100 and 25 years were selected. Design water levels were fixed considering waves, wave run-up and safety margins. Average flow velocities were derived from regime equations, which were 1.48 m/s for 10 yr return period and 1.53 m/s for 100 yr return period. Scour depth was estimated from physical models tests. Depth below initial bed level was taken to be 10.5 m to 11.5 m for a 25 year flood and 10.5 m to 12 m for 100 year flood.

Six different test sections (each varying between 80 m and 100 m) along the main part of the revetment and four test sections along the upstream and downstream termination were implemented over a total length of 660 m. The material for the falling apron varied from section to section, which were: cc blocks of different sizes and arrangements, geobags (250 kg and 900 kg), cc blocks in combination with geobags, gabion sacks, and boulders.

The apron was built in dry condition, on excavated foreshore from +16.30 m PWD to +13.80 m PWD, assumed ground level being in the order of +19.50 m PWD. Launching and falling apron varied in dimension at different location and extended maximum up to 65 m having a thickness of 1.5 m at embankment toe increasing towards river up to 1.8 m at ends. The crest of the embankment is at +22.00 m PWD. The slope of embankment from apron level to crest was 1V:3H.

Damages and maintenance works

Being a test structure, the Bahadurabad Revetment was monitored in a systematic manner right from beginning of construction and continued for next 5 years. Unfortunately, no severe attack and damage of the structure had occurred during the observed period, which could be analyzed for optimized design, although that was the main purpose of the project.

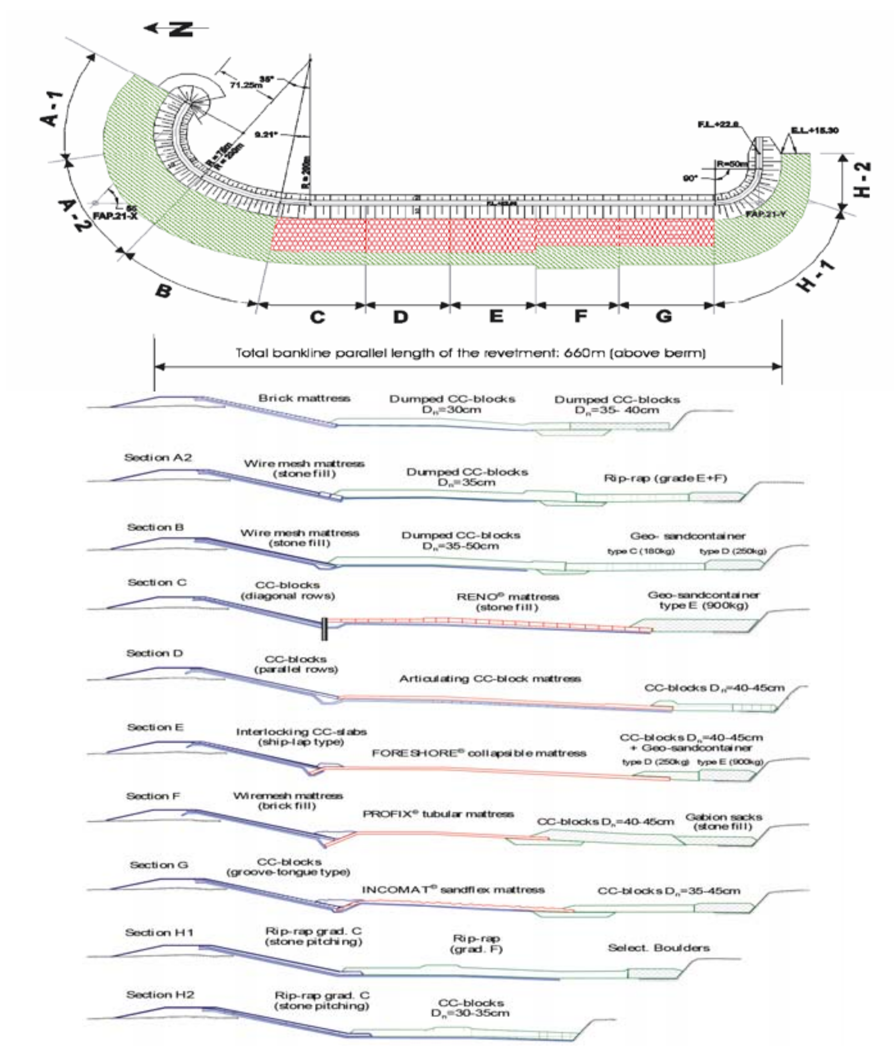


Figure A-10 Layout and different apron materials of Bahadurabad revetment.

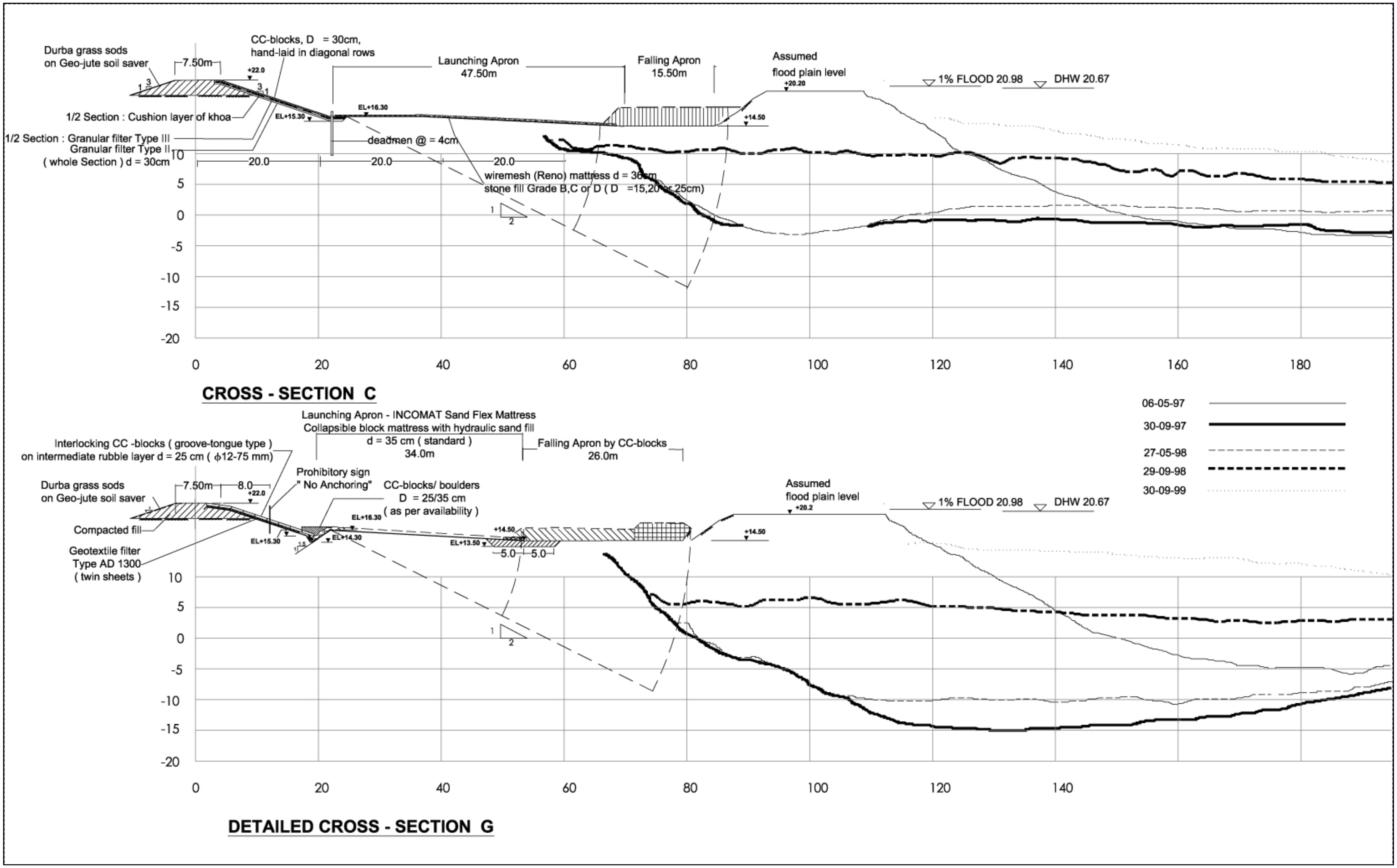


Fig. A.11 Typical cross-section of Bahadurabad revetment.

During 1997 flood, the structure experienced flow attack with velocities up to 4 m/sec. In 1998 flood after initial erosion in the area large deposition took place and since then the structure faces normal flow attack only during monsoon.

Costs

Cost of construction for 800 m long revetment structure was US$ 8.40 million (US$ 10,500 per meter; BDT 446,000 per m; 1 US$ = BDT 42.50). Adaptation and maintenance cost for Bahadurabad revetment from 1997 to end of 2000 concentrating mainly on strengthening the downstream termination point was US$ 360,000. This is 4.25% of the construction cost in three years, i.e., 1.4% maintenance annually. However, it should be kept in mind that the structure didn’t experience any severe attack by the river during the time period mentioned.

Remarks

As a FAP 21 project component, the structure didn’t serve fully the purpose of testing under repeating high loads and subsequent damaging conditions. The site of the structure was selected after a systematic study by expatriate expert morphologists so that the site will be certainly under high flow attack by the river and the structure will defend that by partial failure. However, the morphological changes occurred in different way than that predicted by the expatriate consultants and no important data on responses of all the implemented falling aprons were possible to record. One of the drawbacks of selecting site by the expatriate consultant is that the east bank was morphologically less active compared to the west bank of the Jamuna river. This also indicates the limitations of predictions of river morphology, especially for the Jamuna River.

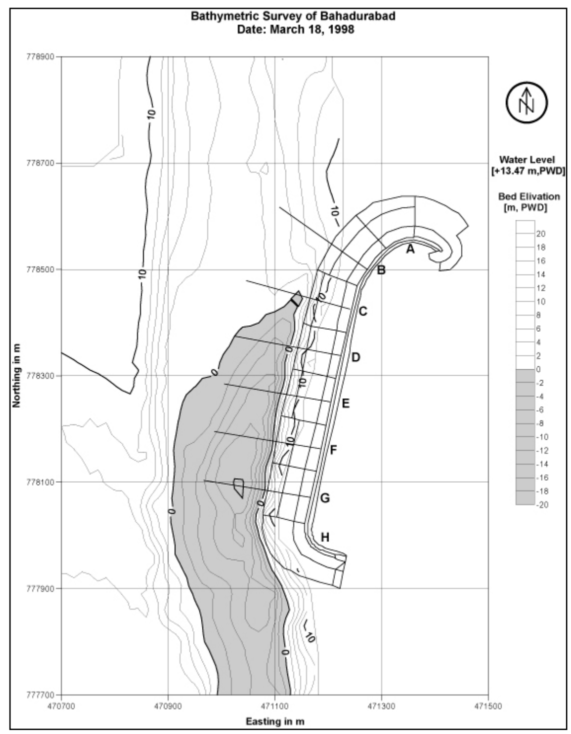


Figure A12: Survey at Bahadurabad Strecture

A.3.7 Ghutail Revetment Structure

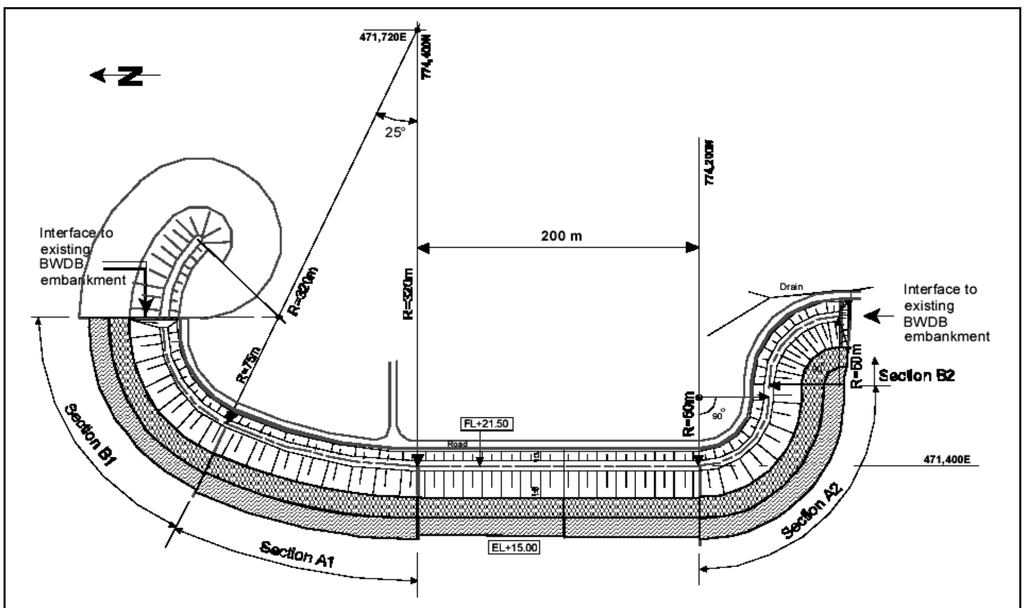
Background

Ghutail revetment structure was constructed about 4 km south of Bahadurabad based on the experience gained from Bahadurabad Revetment. The structure was built with lower safety level due to limited fund, which was actually the remaining budget of the test works. Future extension and completion through BWDB was foreseen.

Design and construction aspects

The structure was built in dry season of 1999-2000**.** The structure is 600 m long with two terminations and protecting about 530 m of bank line. The upstream termination of the structure could not be constructed due to paucity of fund (Figure. A-13 and A-14). The design scour depth in front of the structure was estimated at a level of -5 m PWD. As shown in Figure A-13, brick mattresses were used as a cover layer of the revetment slope (above berm), whereas RENO mattresses and CC-Blocks were implemented as launching and falling aprons, respectively.

Fig. A-13: Construction of the Ghutail Revetment.



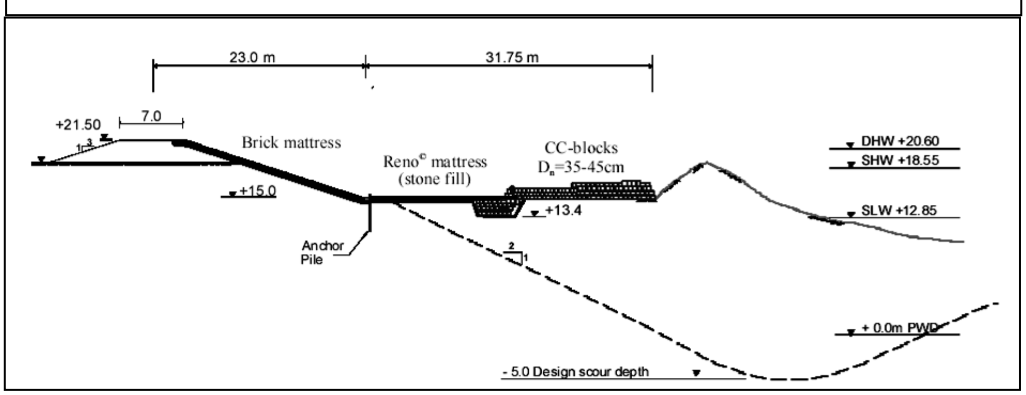


Figure A-14: Layout and typical cross-section of Ghutail

Cost

The construction cost of the structure was US$ 3.56 million i.e., US$ 5930 per meter.

Damages and maintenance works

In contrast with the Bahaddurabad test structure, the Ghutail revetment was subjected to extreme loading due to morphological changes of the river. Scour occurred at downstream part of the structure in the next year of construction, when falling apron was deployed. During 2001-2002 period siltation occurred in the area. After that in 2003 monsoon, scour occurred at upstream side and along the whole structure. The upstream part of the structure was damaged during 2004 flood due to embayment. Sand-filled geobags were dumped in apron and slope to stop failure of the structure. The structure saved a huge settlement area from erosion of the river Jamuna at the back and downstream of the revetment. In early 2005 flood, there was also direct attack and damages of the revetment. After the severe erosion in 2004 and early 2005 flood, the river has a tendency to shift its course away from the structure (Figure A-15 and Figure A-16). A continuous monitoring and observation would be required to adopt the proper repair measure in due time.

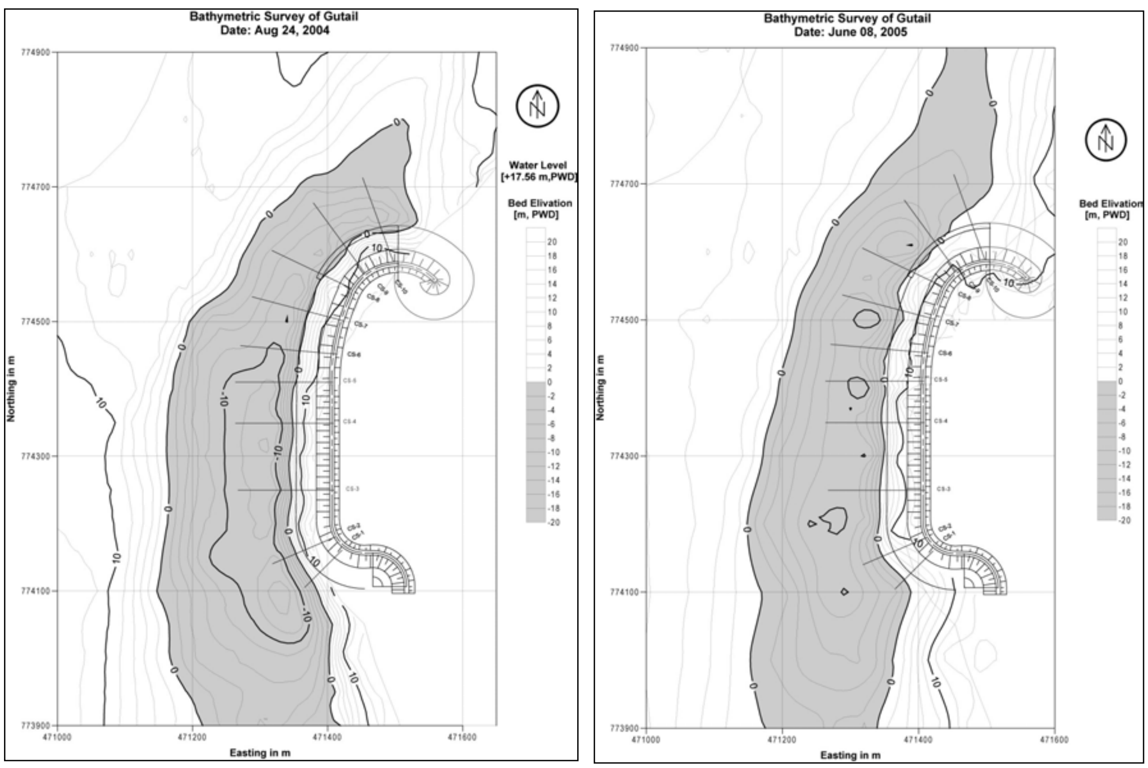


Figure A-15: Bathymetric Survey of Gutail

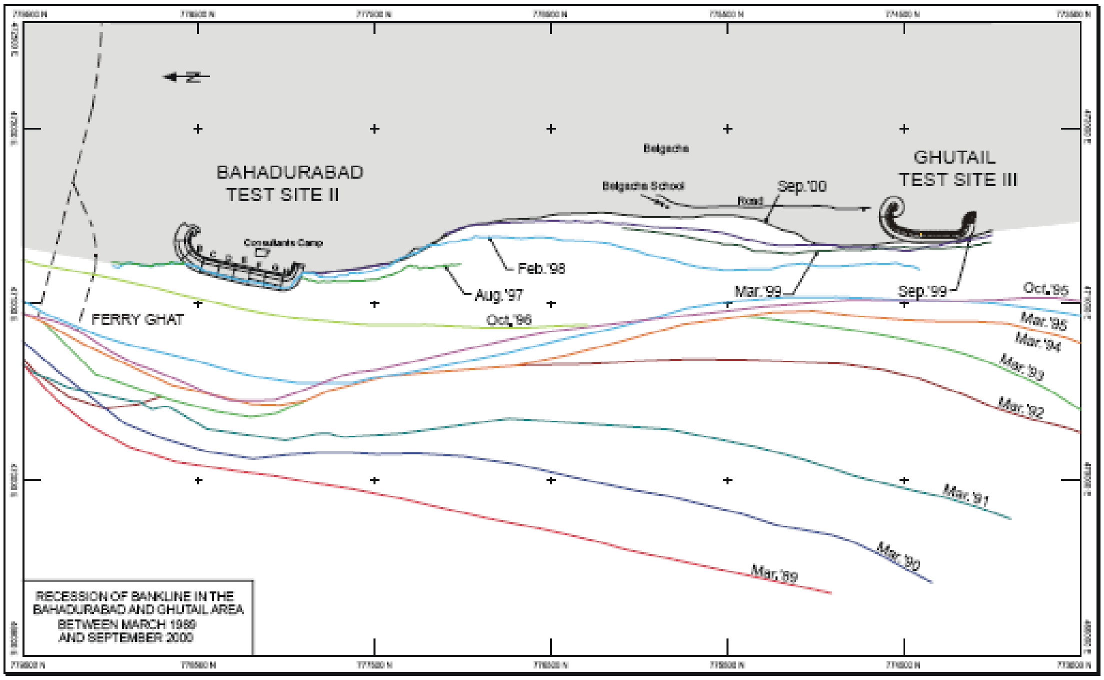


Figure A-26: Bankline of Gutail and bahadurabad.

Remarks

For the revetment, design scour depth was much smaller than observed scour depth in front of the structure. The monitoring data of the structure shows some behavior of the launching apron under extreme attack. It appears that for the same shape of materials (e.g., cc blocks in this case), the launching performance are not significantly affected by size of the material. For example, almost same performance of the 35-40 cm blocks and 40-45 cm blocks was observed. As the scour hole develops, the slope with deployed apron materials approaches towards a slope of 1V:1.5H. With repetitive and angular flow attacks, the slope normally attains the maximum value. There were mainly two reasons for the damages to the test structure. The first reason was the incomplete upstream termination. Other is the excess scour depth than the design condition causing shortage of apron materials to be deployed. This finally led to a flattening of the slope. The voids in the protective layer got bigger and sand was washed out until the entire slope disappeared.

A.3.8 Chandpur Town Protection

Background

The classical example of fighting nature with very limited resource is the protection of Chandpur Town. Located on the left bank just downstream of the confluence of Upper Meghna River and Padma River, the town is exposed to the full fury of the combined flow of the Padma and Meghna during high monsoon. Additionally, tidal fluctuation, wave action and the presence of the Dakatia River outfall have contributed the area very vulnerable to erosion.

Protection works

Railway department used to dump boulders in the past to protect its railway head at Natun Bazar (Northen part of the town). Due to the activity of the channels in the Padma-Upper Meghna confluence area, the erosion intensity in the Chandpur area periodically becomes severe. In such situations measures were undertaken on emergency basis. Following serious erosion in 1970, large amount of boulders were dumped on the Puranbazar site. A 96 m long groyne was constructed in 1972-1973 at Puran bazaar using about 80,000 m^3 boulders. In the next flood, a part of the head of the groyne failed by material slides. Due to lack of repairs, further damages and slides occurred in 1974 flood. The groyne was abandoned but the remaining submerged part created excessive local scours in the area. In the following years revetment works have been carried out in the area with boulder riprap. A comprehensive plan in 1975 recommended a total 540,000 m^3 boulders for 2408 m bank protection, i.e. 224 m^3 per meter. Due to resources constraint, only 27% of boulders were dumped by 1988. The rate of dumping of boulders was slow, about 11,150 m^3/year. The amount of boulders was inadequate per meter length and also sizes were not always according to the recommendation. After 1988, cc blocks were used instead of boulders. Since 1990, revetment works have been carried out with different sizes of cement concrete blocks over sand filled geo-textile bags. Earlier works mainly consisted of stone rip-rap over khoa and sand filter. By 2003, about 2.5 km of revetment has been constructed along the Meghna and the Dakatia River.

Cost

The town is being protected on contingency plan from early seventies. From 1972-73 to 2005 an amount of about BDT 110 crore has been spent to save the town and its adjacent bankline for a length of about 2.90 km.

Remarks

The bankline at Chandpur has been almost fixed for the time being but erosion at the upstream and downstream reaches is continuing. The town still remains vulnerable to erosion as before. Currently, the deepest scour hole in front of the Mole head (small protrusion just upstream of outfall of Dakatia River) is about 70 m from high water level. The revetment slope in that locality is mostly 1V:1.5H or even steeper at some locations. Proper monitoring is essential before taking any further measures around Chandpur town.

A.3.9 Rajshahi Town Protection

Background

Rajshahi Town protection is a good example of long-term protection activities to save the town from the erosive attack of the Ganges. The average level of Rajshahi Town is below the normal flood level of the Ganges. So flood protection embankment was constructed as early as 1855. However, the protection against erosion started long time after that. Especially, the training measures in the Indian part of the river has resulted shifting of the Ganges in the downstream reaches and thus bank protection became an important issue in Bangladesh part as well.

Design and construction of protection works

In Rajshahi Town protection scheme almost all types of bank protection works have been practiced**.** In 1960, 6.5 km of the embankment was implemented with bricks mattress encased in 2-layers of galvanized iron wire net. In 1970, brick-filled gabions were used to save the flood protection embankment from flood damage. At that time 2 spurs at GPO and Dargapara were constructed on emergency basis. After the independence in 1971, up to 1977, a series of brick-crate spurs and a T-groyne (Groyne No. 1) was constructed. After construction of these structures, the area downstream of Police line was safe but upstream reach was experiencing serious erosion. Consequently, two groynes (Groyne No. 2 and 5) and a brick-crate spur (No. 2B) were constructed by 1980 after conducting physical model study. These structures with brick mattress revetments protected the Rajshahi Town for several decades with required repair-maintenance works after floods.

Following the 1987 flood, 5.8 km revetment works and seven groyne or spurs were rehabilitated and constructed. Again, in 1998 flood, considerable damages occurred in the protection works of Rajshahi, which were repaired under the funding of 1998 Flood Damage Rehabilitation program. Among others Groyne 1 and Groyne 2 were rehabilitated and 2 km revetment works were carried out. For slope protection, cc block (40cm x 40cm x 20cm) revetment over geo-textile filter with cc blocks as toe protection were used in these structures. A RCC spur was built at Panchabati (Shashan ghat) area in 2000-2001 costing about BDT 5.5 crore. This spur does not have an earthen shank. The length of the RCC part is only 70 m. The bed protection is provided by 35 cm and 45 cm cube blocks and boulders. Some photos of Rajshahi town protection works are shown in Figure. A-17.

Currently, works are in progress to protect bankline between the Groyne-1 to the RCC spur with revetment mainly composed of sand-filled geotextile bags with cc blocks.

Cost

Total construction and maintenance cost from 1973 to 2003 for all the components constructed for the Rajshahi Town protection is about BDT 86.23 crore.

Up to 1996, the cost of Rajshahi Town protection was BDT 63 crore (BDT 39,000 per meter) in 1996 price. This includes 1987/88 flood damage rehabilitation (FDR) cost, which is about BDT 9 crore in 1996 price. The 1998 FDR cost was BDT 47.5 crore. That means the flood damage rehabilitation cost for two major floods of 1987/88 and 1998, exceeds the total amount spent without FDR from independence to 1996. On average, maintenance cost is 5.5% of the initial cost of construction of protection works during the period 1982 – 2000.

Remarks

Brick mattresses were effective for long time with regular maintenance but didn’t sustain in large floods. In the original design, no proper launching apron was provided. The revetment work consisting of brick mattress proved to be unsustainable due to damage of wire net and consequent loss of bricks.

Figure A-17: Structures for Rajshahi Town protection: RCC spur and cc block revetment (above) and T-groyne head with brick mattress (below).

A.3.10 Protection in Bhola

(i) Bhola Town Protection

Bhola Town and its adjacent area, Tulatali Bazar, and severely affected bankline in Doulatkhan and Tajumuddin Upazilla were provided protection through construction of revetment. Cement concrete blocks (40.32 m^3 per meter) were placed in the trench along the bankline over a width of 28 m. The blocks launched with the advancing scour and arrested the erosion in the affected reach.

(ii) Protection in Tajumuddin

Design and construction aspects

Protection with sand-filled geobags has been provided in Tajumuddin Upazilla for a length of 2277 m in Lower Meghna River. The bankline above 0.00 m PWD to +3 m PWD (floodplain level) were protected with 450 x 450 x 300 mm^3 cc blocks over a geotextile filter. Sand filled geobags were placed and dumped from 0 m PWD to -21 m PWD. Maximum scour anticipated in the design was up to -30 m PWD. Total volume of material (geobag) dumped per unit length of bank line is 50.00 m^3 per meter. Two different sizes of geo-bags weighing 175 kg and 126 kg were used.

Another 887 m of river bank, just upstream of the above protection was constructed with 50 m^3 per meter of cc block from 0 m PWD to -21 m PWD. Two different sizes of cc blocks, 350 mm cube 50% and 400 mm cube 50% were dumped. The slope above low water level from 0 m PWD to +3 m PWD was protected by one layer of 450 x 450 x 300 mm cc block over a geotextile filter. The anticipated design scour in this case was also up to -30 m PWD.

Damage and maintenance works

No damage has so far been reported. Monitoring survey conducted on the protected reach and in the vicinity (during May–December, 2005) shows that maximum scour along upstream and downstream end of the revetment are around -32.50 m PWD. In other areas along the revetment the bed level are in the range of -15 m PWD to -22.50 m PWD.

Cost

Cost per meter of protection at the downstream site (using geobag as underwater protection) was BDT 75,582 resulting in a total cost of BDT 17.21 crore for 2277m protection work. Cost per meter protection work of the second site (using cc blocks as underwater protection) at upstream was BDT 148,250 and total cost was BDT 13.15 crore for 887 m protection work.

Remarks

The protection work appeared to be effective. A continuous monitoring is needed to observe the behavior of the protected reach and also the morphological changes of the river.

A.3.11 JMREMP

Background

The Jamuna-Meghna River Erosion Mitigation Project (JMRMEP) was formulated to control riverbank erosion at two major Flood Control Drainage and Irrigation (FCDI) projects, namely the Pabna Irrigation and Rural Development Project (PIRDP) and the Meghna-Dhonagoda Irrigation Project (MDIP). The duration of pilot phase of the project was from 2004 to 2006.

The feasibility study of the project recommended for using revetment incorporating a falling (launching) apron consisting of loose material. In the interest of reducing the cost, *geobags* were considered for this purpose. The feasibility study also suggested an *adaptive approach* for managing river erosion by revetments along the natural alignment of the river system using sand-filled geotextile bags at launching apron. It is an adaptive approach, which means that interventions are planned according to observed and predicted river behavior. Extra layer of launching materials are dumped from the river in a controlled manner as and when necessary, which is determined by organized monitoring.

Design and Implementation

Considering maximum expected scour (30 m for PIRDP and 20 m for MDIP below low water level), suggested apron protection in PIRDP and MDIP is 60.80 m^3 per meter and 40.66 m^3 per meter respectively. The amount of material is computed by estimating the maximum scour assuming a 1V:2H launching slope. In the first year (pilot phase), the launching length covered was 33.5 m. With 0.61 m as launched material thickness, 20.45 m^3 of material was required to cover the slope.

(i) PIRDP

The 2002 feasibility study suggested protecting about 7 km length of vulnerable bankline that extends from the outlet of the Hurasagar River in the north to Kaitala Pump House in the south (Figure. A-18). Under the project, two reaches of the bankline have been identified as susceptible to erosion of different severity. To accommodate adaptive approach, the activities are phased over six year period. The reach is selected according to the existing geometry of the riverbank.

In an attempt to save Pechakhola and Mohonganj area from severe erosion in October 2004, a programme was undertaken to provide protection in 600 m bank line, 200 m in Pechakhola and 400 m in Mohanganj. The original estimate was to dump 60 units of geo-bags per meter (20.16 m^3 per meter). During actual execution volume of dumping material was adjusted as per site condition. In the changed circumstances 2225 linear meters were covered by mass dumping of geobags in seven categories of units ranging from 20.16 m^3 per meter to 3.25m^3 per meter. All these adjustments were done to provide the protection as per requirement at that time making the best use of available materials. Figure A-19 and Figure A-20 show implemented works at the PIRDP as per the concept of adaptive approach.

Under this project, 1200 m long grout filled mattress for wave protection has been built for pilot test purposes. The mattress work has been implemented during the dry season 2006-2007.

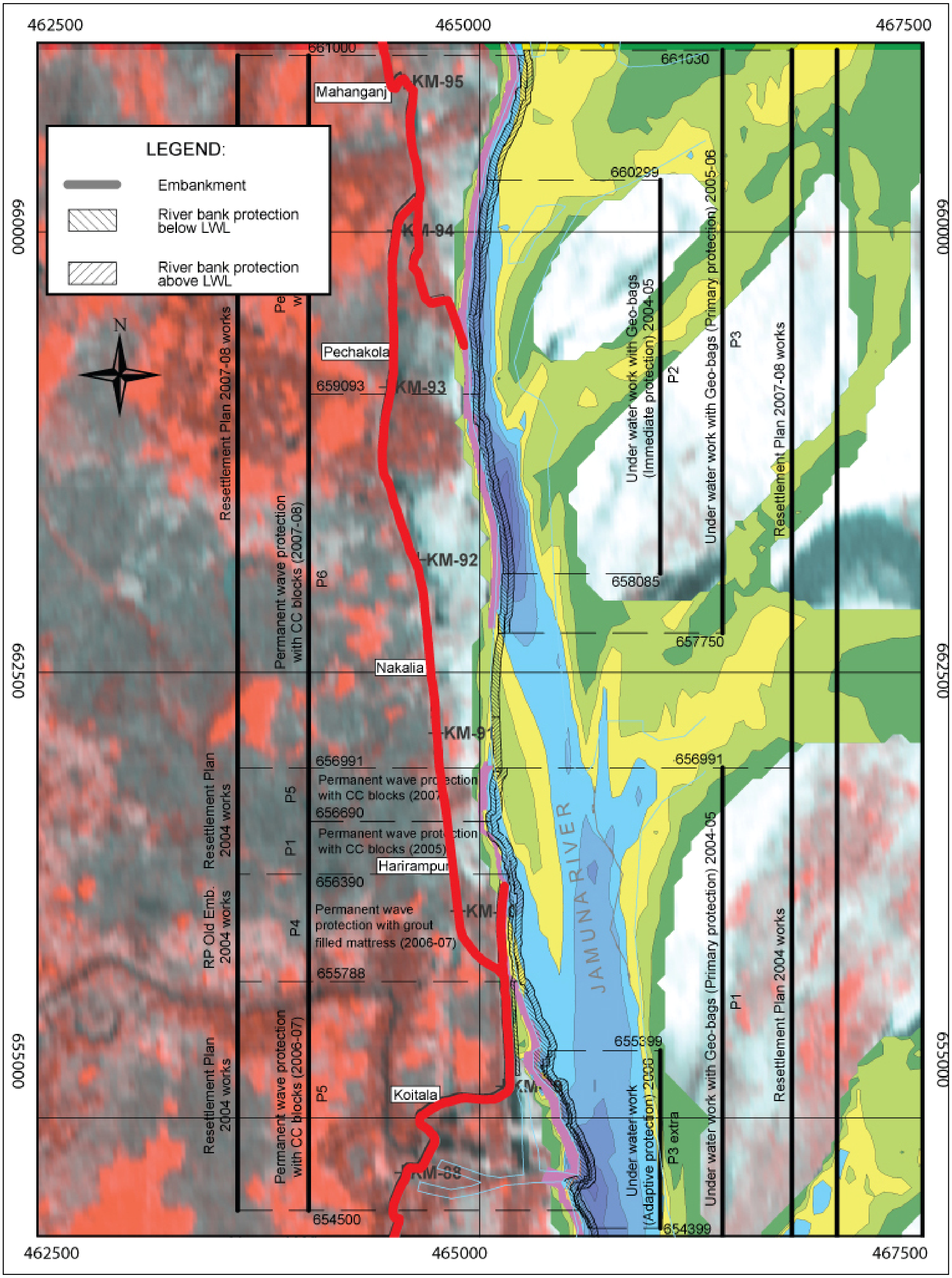


Figure A-18: Site map PIRD

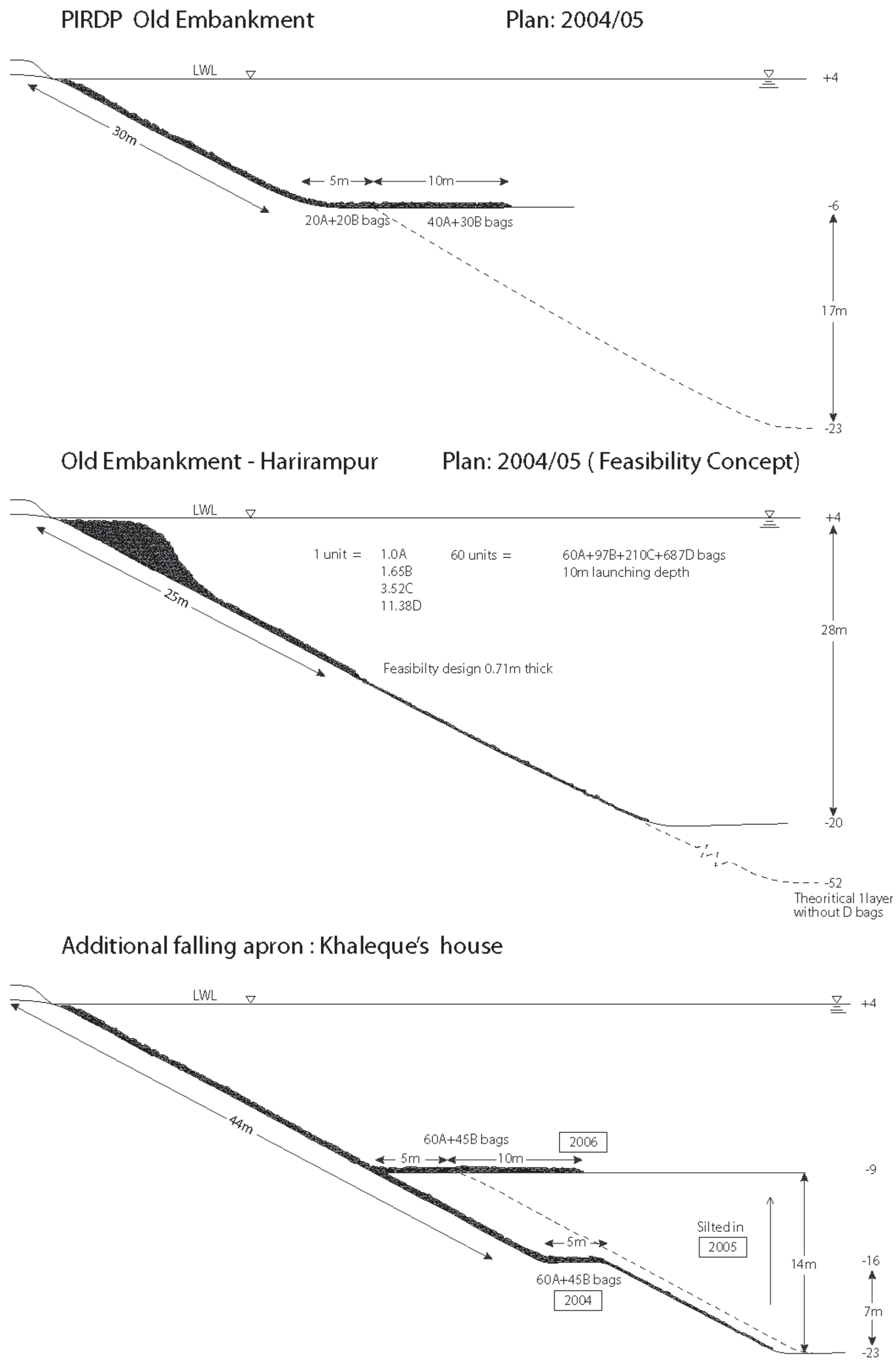


Fig. A.19 Implemented work at the PIRDP as per principles of the adaptive approach.

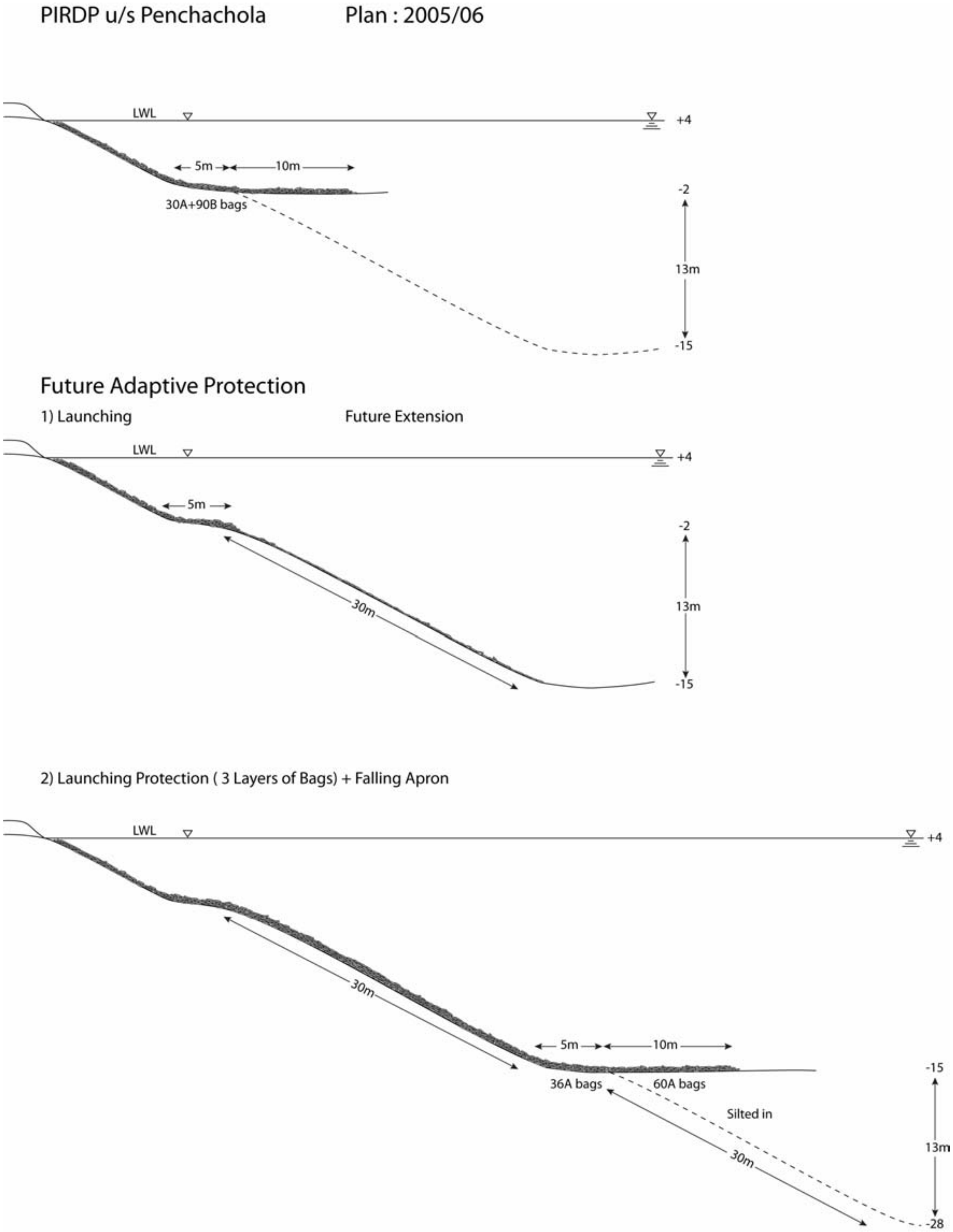


Fig. A.20 Implemented work at the PIRDP as per principles of the adaptive approach.

(ii) MDIP

The site is under the influence of variation of channel pattern of Padma River. The process of bank erosion consists of two distinctive elements: (a) wave erosion at the surface that results in immediate loss of flood plain and (b) river erosion during flood season which results in gradually steeper slope of the bank**.** Based on 2002 feasibility study, the morphological analyses identified two distinct reaches of the bank from Ekhlaspur to Dasani being under threat. A reach of about 4.4 km length was recommended for implementation initially as Phase-I. Based on the future point of river attack, the two sub-reaches were identified later, to take up one of the sub-reaches under Phase-II for early protection. Another reach may be identified under Phase-III. Stage-II and III provide two increments of revetments, each 2.2 km in length. Figure A-21 shows implemented works at the MDIP following the adaptive approach. Figure A-22 shows a typical cross-section of the Dosani revetment.

Under the project in MDIP, 1250 m of bank from km 44.98 to km 48.23 was provided protection against erosion by fascine mattress ballasted with geo-bags, falling apron and area coverage by geobags. Ekhlaspur hard point (Figure A-23) from km 48.230 to km 48.330 (100m) has been rehabilitated by cc blocks in apron.

In 2002 and 2003, different reaches of bank within the selected area were given protection with different quantities of geobags. All these type of protection applied so far have different dimension at different reaches as per site condition (e.g., Figure A-21). These are monitored systematically for further improvement or rectification which is the essential part in the adaptive approach.

Cost

Cost per linear meter of launching apron in the PIRDP is BDT 106,000. Including the slope protection with cc blocks and geotextile filter from +2.00 m PWD to +9.50 m PWD, the total cost per linear meter of protection was estimated to be BDT 146,000. This cost estimate accounts for protection work undertaken in the phased programme during 2002 to 2005.

In the MDIP, the cost of apron protection (2002-2005) and temporary protection against wave action is about BDT 84,000 per meter length of bank. Including the slope protection with cc blocks over geo-textile filter from -1 m PWD to average bank level the total cost per meter of protection becomes BDT 119,000.

Remarks

For the protection in two sites an adaptive and phased approach, being the basic concept of the project, has been followed in selecting type and volume of materials. A typical design section for geobag revetment set under this project is shown in Figure A-24 and Figure A-25.

The major achievement was the successful protection of the flood embankments at both sub-projects before the exceptional 2004 flood. In total 7.06 km of riverbank protection up to flood plain level (FPL) and 1.12 km of slope protection up to High Flood Level (HFL) in PIRDP and 4.00 km of protection in MDIP was built to a first level of protection by July 2004 and survived the flood season without any significant damage.

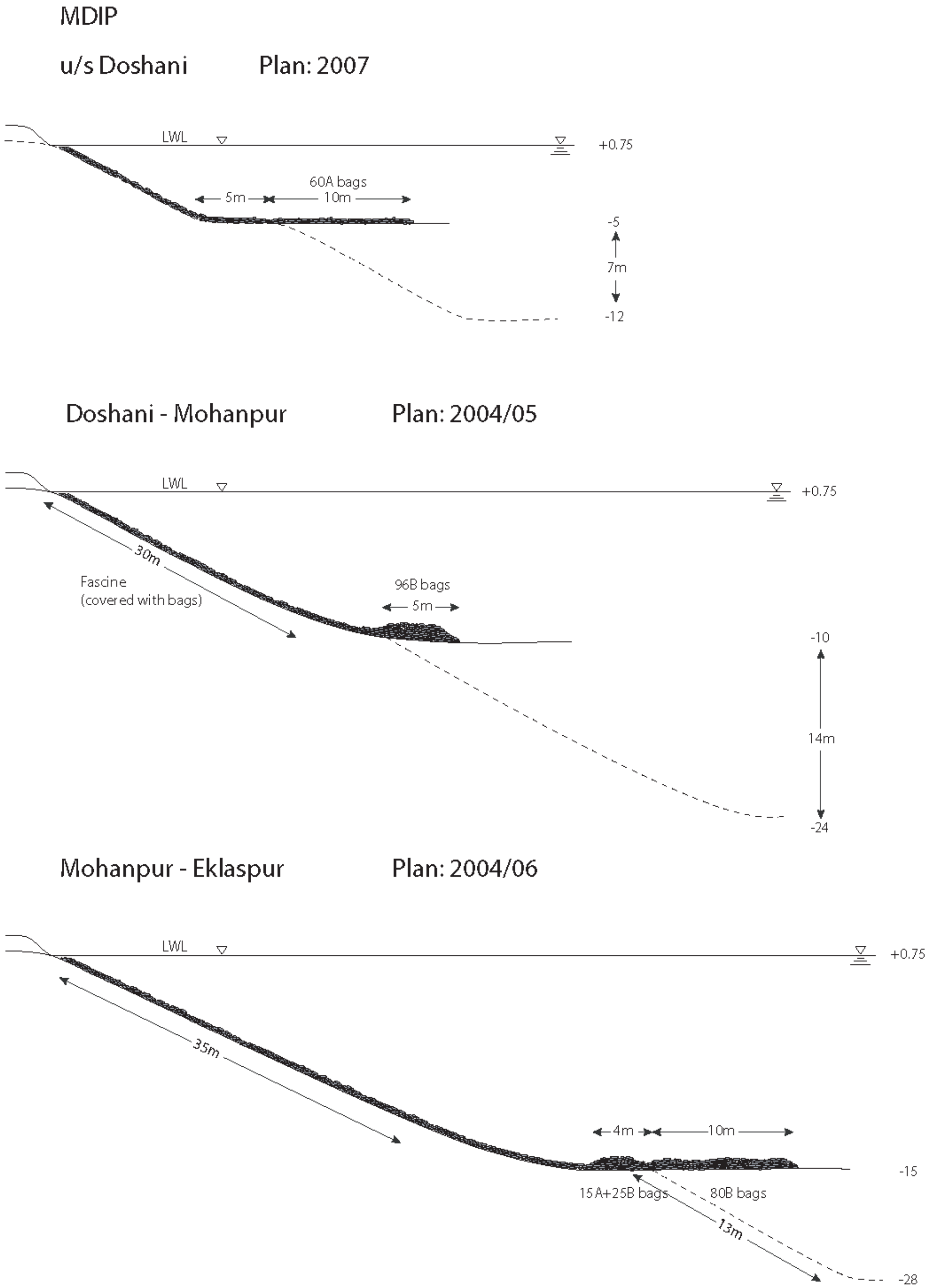


Fig. A.21 Implemented work at the MDIP as per principles of the adaptive approach.

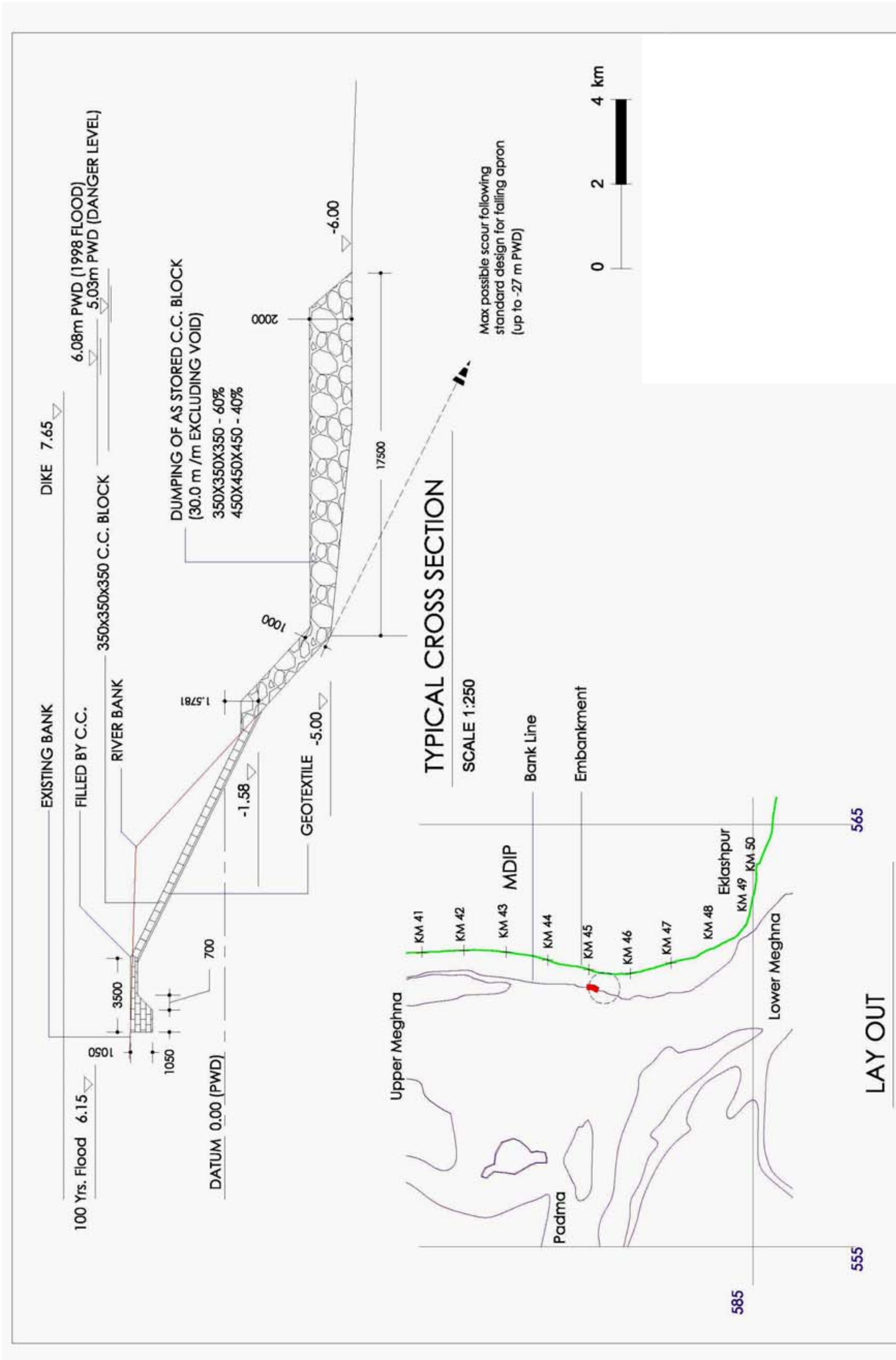


Fig. A.22 Layout and typical cross-section of Dosani Revetment.

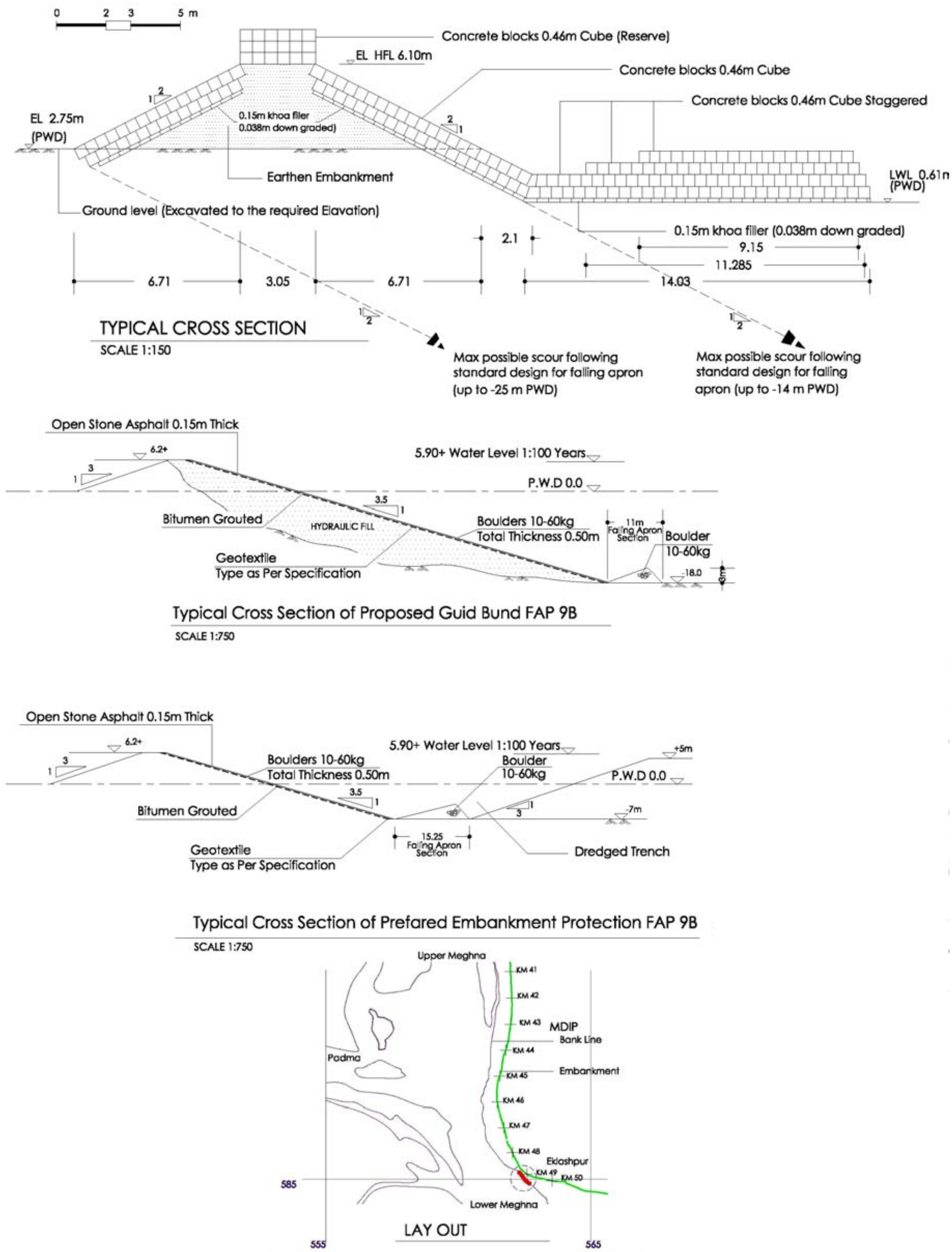


Fig. A.23 Typical cross-section of the Eklashpur Hard Point.

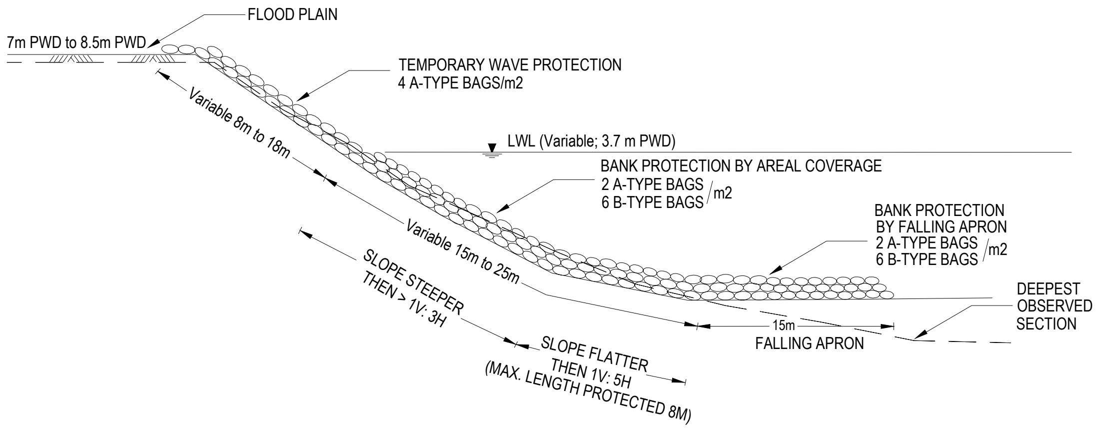


Fig. A.24 Design for riverbank and temporary wave protection as adopted in the JMREMP (PIRDP).

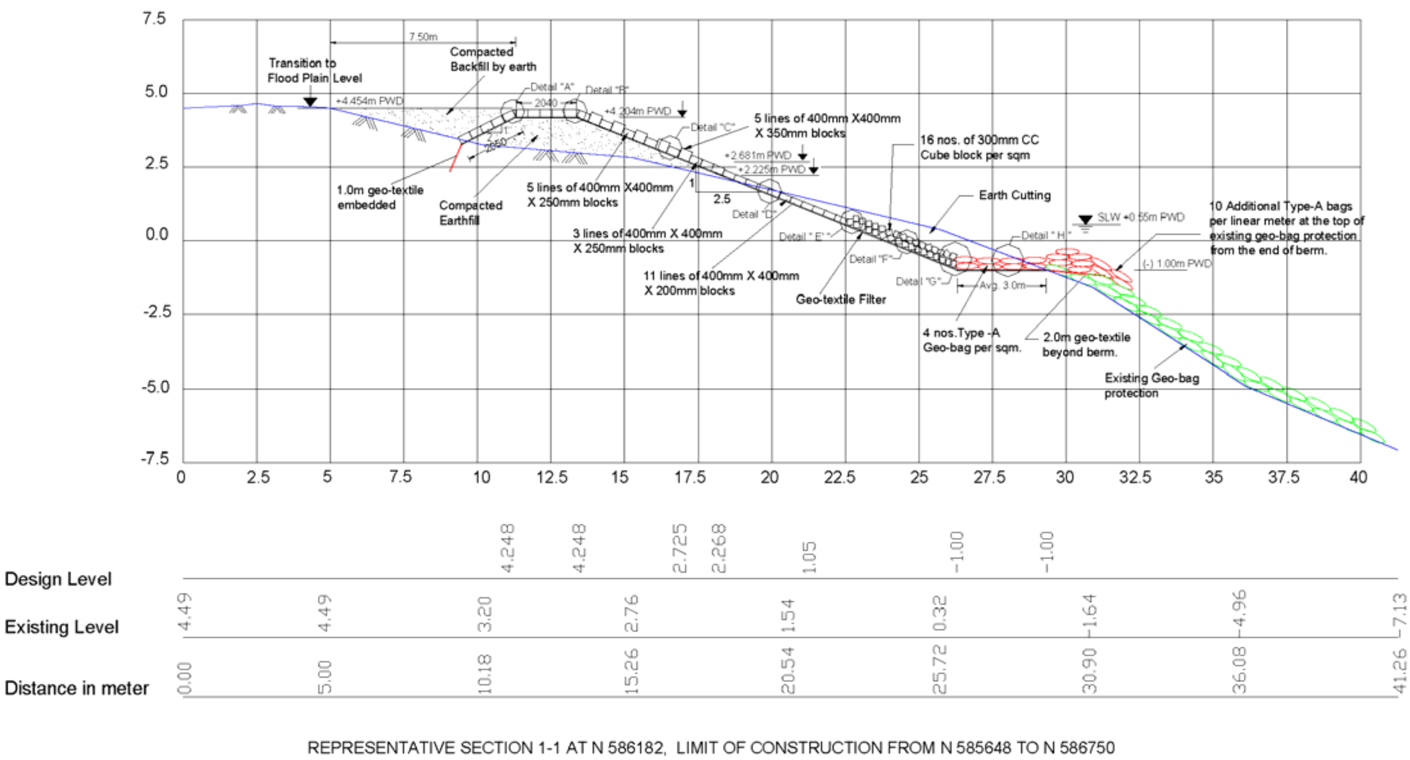


Fig. A.25 Design for permanent wave protection as adopted in the JMREMP (MDIP).

A.4 Groynes and Spurs

A.4.1 Kalitola Groyne

Background

Kalitola groyne was built by BWDB during mid 1980s at 117.5 km of the Brahmaputra Right Embankment in Bogra district. Under the River Bank Protection Project (RBPP), the groyne was rehabilitated in 1998. As mentioned earlier the purpose of Kalitola Groyne in combination with Sariakandi and Mathurapara hard points was to prevent Jamuna River from merging with the Bangali River that poses the risk of bypassing the Jamuna Bridge.

Design and construction aspects

The layout and cross-section of the groyne are shown in Figure. A-26. The length of the main groyne part is 111 m with a cross bar of a length of 134 m. Following are the salient points for design:

High flood level = 19.77 m PWD (1 in 100 yrs flood)
Low water level = 11.43 m PWD
Design discharge = 76, 500 cumec
Design velocity = 4.8 m/s (nose of groyne); 3.7 m/s (shank of groyne)
Scour depth = 33 m depth or -20 m PWD (nose); 29 m depth or -14 m PWD (shank)
Apron level = +0.43 m PWD ; Apron width = 20.5 m (nose); 14.5 m (shank)

Damages and maintenance works

Kalitola groyne was completed in 1998. In September of the same year the scour level reached to -7 m PWD and the apron was deployed. In the next June, the BWDB O&M unit dumped 1500 no, 450 mm size cc blocks to replenish the deployed apron blocks just downstream of the nose. On 6th July the first damage of the spur-head occurred, which was countered by dumping cc blocks (650 mm size) and geobags (2.3m x 1.15 m) until 25th July. The minimum scour level was -12.00 m PWD in July 10. By mid-August further damages occurred and the cc blocks on the revetment near the water surface were undermined on the upstream side of the nose.

The measurement in September 2000 showed a scour bed level of -14.50 m PWD. During 2000 flood more than 100,000 geotextile bags of different sizes were dumped. In the next low water season the slope protection and falling apron were rebuilt.

On September 4, 2007, 145 m of revetment at upstream face of shank and on October 23, 2007 another 125 m of bell mouth face was damaged creating a vertical face at the end, corresponding WL was 16.25 m and 13.84 m, PWD. Again on November 27, 2007 another 75 m of upstream face of the bell mouth was damaged, corresponding WL was 11.53 m, PWD. The recorded scour in the vicinity of the structure was (-) 22.10m, PWD on 22.08.07 at a distance of about 130 m, (-) 35.90 m, PWD on 22.10.07 at a distance of about 160 m and (-) 20.70 m, PWD on 24.11.07 at a distance of about 110 m from the crest wall.

Cost

The initial cost of construction of the Kalitola groyne was US$ 7.4 million (BDT 31.6 crore; 1 US$ = BDT 42.7). The cost of rehabilitation in the next low water season was more than BDT 100 million. The cost of dumped geotextile bags in 2000 flood season was about BDT 20 million.

Thus the total repair cost of Kalitola Groyne was about BDT 120 million (US$ 2 million, 1 US$= BDT 60) after construction, which was about 30% of the initial construction cost.

The damages that occurred in September to November, 2007 were repaired in the next dry season during March-June, 2008 with 45 to 50 cm cc blocks (40 m^3/m to 50 m^3/m at launching apron) and cc blocks on slope over a geotextile filter. The cost for the repair due to damages in 2007 was BDT 96.627 million.

Remarks

The reason for failure of the Kalitola Groyne was mainly the rapid deep scouring with subsequent development of a sequence of flow slides. The falling apron was unable to react rapidly to the flow slide's penetration underneath the protection. The failure mechanism was similar to the Sirajganj hard point. Low relative density of micacious sand in the Jamuna is the main geotechnical factor in this regard.

The failure occurred in 2007 was also repetition of the events responsible for earlier failures. This time the constricted flow due to formation of a char at the opposite near bank was the primary reason for inducing the scour.

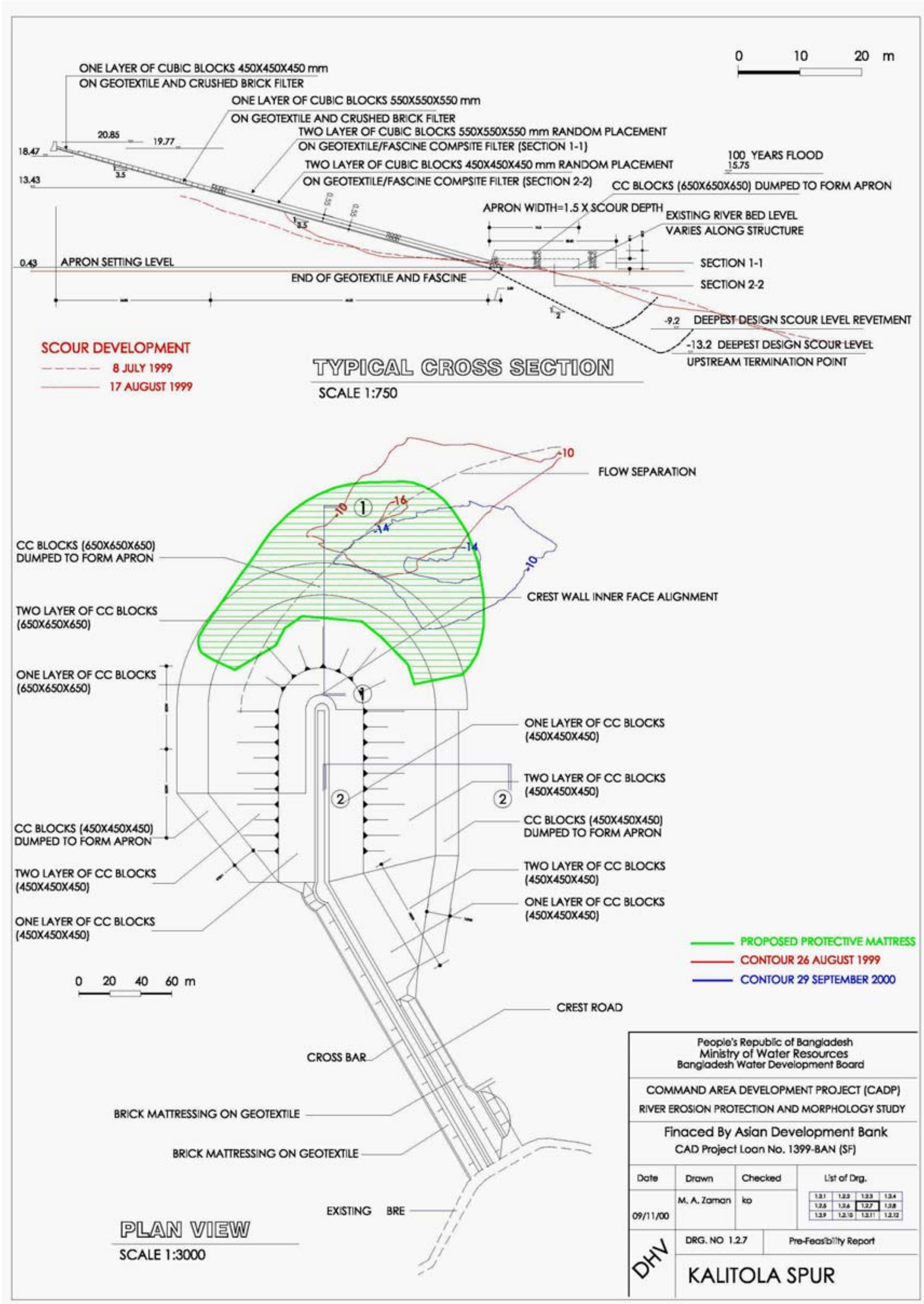


Figure A-26: Typical cross-section and layout of the Kalitola Groyne.

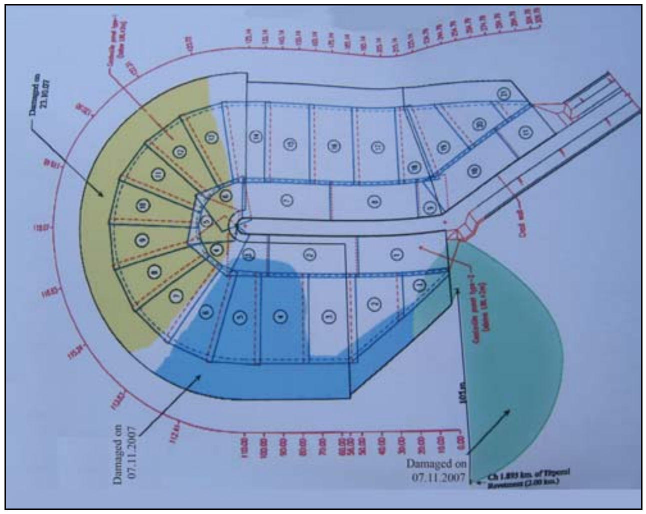


Figure A-27: Damage of Kalitola Groyne in 2007

A.4.2 Sailabari Groyne

Background

Sailabari Groyne was constructed in 1978 under Sirajganj O&M Division, BWDB along the right bank of Jamuna River (BRE Km 162.300) at a place named Ziar Morh. Initially, the groyne fulfilled its purpose and protected nearly 2 km of bankline from upstream of the groyne to the Sirajganj town located downstream of the groyne. The structure has recently been completely destroyed.

Design and Construction Aspects

The design of Sailabari Groyne can generally be described as an earthen dam with a slope protection like revetment. The side slopes and head of the groyne are protected with cc blocks. As usual the cover layer, filter layer and falling apron are used.

The groyne as constructed had a 1 km long earthen shank with a 60 m T-head. Revetment on slope of the shank, the T-head and toe protection required for the anticipated scour were provided based on the existing and expected bathymetry and morphological condition.

Damages and maintenance works

The main flood bearing channel in the vicinity of the Sailabari groyne started to shift towards western direction in 1994 and consequently severe erosion started along the right bank during monsoon in those days. The groyne faced a major attack from the river in 1996 due to continuous shifting of the main channel towards the west bank at the upstream of the groyne. As a result, 20 m of the T-head was damaged. Due to unfavorable conditions the damaged T-head part was not restored but to arrest progressive upstream embayment and thereby to protect the groyne against further damage, a 188 m long revetment work was constructed at upstream of the groyne in addition to major repair of the T-head.

During 1998 flood, the T-head and a portion of the shank was severely attacked by flow. To protect the groyne and T-head, 9000 m^3 of cc blocks were dumped. After 1998 flood, to support the groyne, 680 m revetment was constructed at upstream side and 450 m^3 of cc blocks were stockpiled for emergency.

In May, 2002, water level in Jamuna River started rising and oblique flow developed towards Sailabari Groyne. On 28 June 2002, another 20 m out of the remaining 40 m T-head part were washed away. Following this, 25 m of the shank along with its slope and launching apron disappeared. On 26 July 2002, another 20 m of the shank was destroyed and the T-head was detached from the shank. Within a few hours 105 m shank was totally damaged. The damaged groyne was again reconstructed during 2002-2003 (Figure A-28).

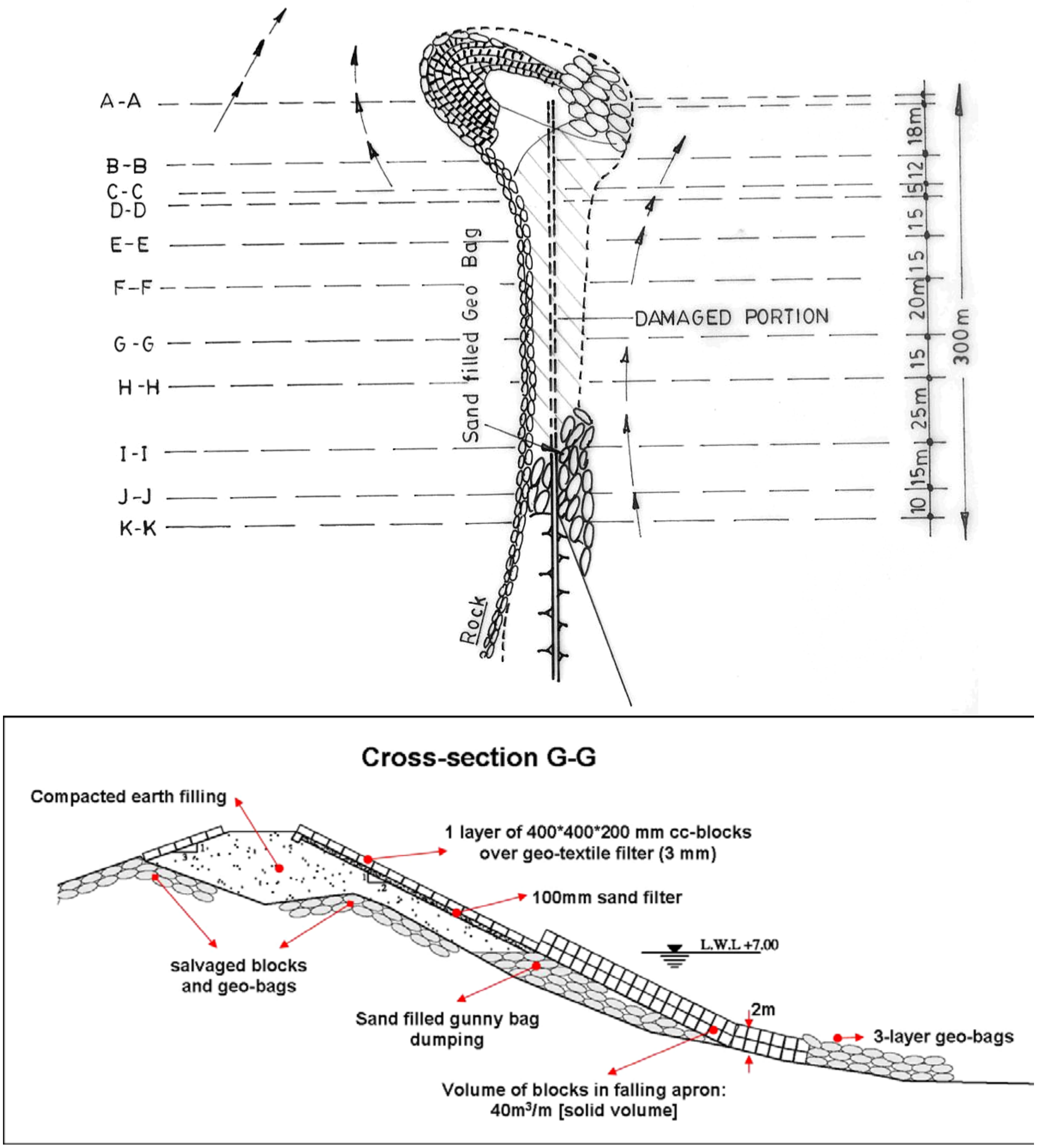


Fig. A.28 Reconstruction of Sailabari groyne in 2002.

On 19 July 2003, about 27 m of the rehabilitated shank was suddenly damaged. At that time attempts were made to control the situation by dropping gunny-bags, geo-bags and cc-blocks. During receding water level in Jamuna River, a slip circle was developed along 105 m of the shank and subsequently settled into the river. Again attempts were made to protect the remaining part of the shank by dropping geo-bags and cc-blocks. At that time the river was flowing 0.72 m below the danger level (14.00 m PWD). The groyne lost its function after 17 October, 2003 when it almost totally washed away. In that year, 430 m out of the 600 m bank revetment, constructed initially to protect the groyne, disappeared in the river.

During 2004, attempts were made to protect the remaining part of the groyne. While the repair work was in progress (about 13% progress), the remaining parts of T-head and shank disappeared into the river. Subsequently, during 2004 and 2005 monsoon, remaining 340 m shank of the groyne also engulfed into the river, ending this long process of repair and reconstruction.

As per satellite image of January 2006, the position of Shailabari groyne where it existed is now within the river, about 1 km from the west bank.

Cost

The initial cost of construction in 1978 was BDT 7.50 crore. The data for 1996 damage repair is not available. Cost of repair-maintenance works in the year 1998, 2002-03 and 2004 were BDT 8.5, 7.52 and 2.24 crores, respectively. Thus the rehabilitation and maintenance cost for 26 years appears to be much more than the initial construction cost without the cost data of 1996.

Remarks

The main reason for complete failure of the Sailabari Groyne is morphological changes of the river and the upstream embayment of the main channel. The apparent reason of the destruction in 2002 can be referred to high turbulent eddies occurring in front of the groynes at the downstream part. The deep scour hole was generated by high turbulent eddies occurring near the head of the groyne due to deflected currents from a char near the groynes.

Nearly every scour hole occurred slightly downstream of the tip of the groyne as it supposed to be for a single attracting groyne. Observed maximum scour depth on the 26th July 2002 was 49.65 m from the 100 year flood level (+16.35 m PWD) when the structure was damaged. On October 21, 2003, the maximum scour depth of 56.75 m was observed. The slope of the scour hole in front of the groyne head during these high scour depths were steeper than 1H:2V.

Even the gentlest observed slope of 1V:1.9H is steeper than the critical one of 1V:2H, referred to "*Determination of Stable Slope for Deep Riverbed*" out of the *Geotechnical Investigation (2003-2004)*. The steepest observed slope of 1V:1.2H is situated close to the groynes head and appears twice in July 2002 and May 2004. After the failure in Oct. 2003, the minimum bed level rapidly rose by 25 m in 50 days due to progression of a char in the area.

During the reconstruction of some parts of the damaged groyne, cc blocks of almost uniform size were used as falling apron. This type of cc block riprap has the disadvantage that rapid winnowing of the base materials occurs under very turbulent flow condition.

The next *Figure A-29* is a map of the area, where Sailabari Groyne used to be located including the shifting bank line from 1997 to 2006. This map visualizes the problem of outflanking currents with a real situation. The char situated east of Simla Spur 3 together with the char which now includes Sailabari groyne developed a current that directly pointed on the upstream bank. This process of development can be seen in Figure A-30 where the satellite pictures from 2002 to 2005 are shown. The situation caused tremendous erosion and shifted the bank from 2004 to 2006 approximately 1km behind the groyne.

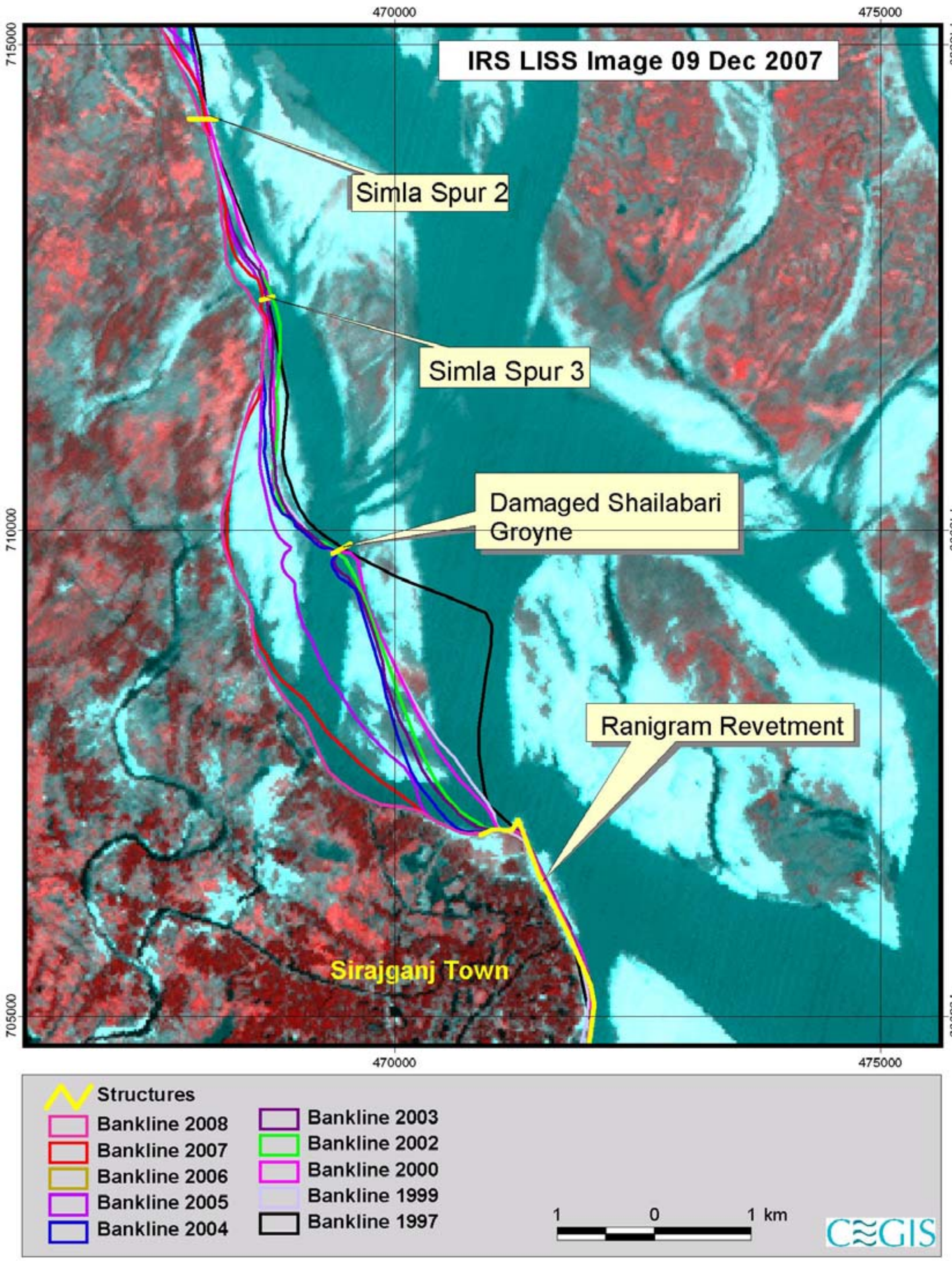


Figure A-29: Bank line near former Sailabari Groyne

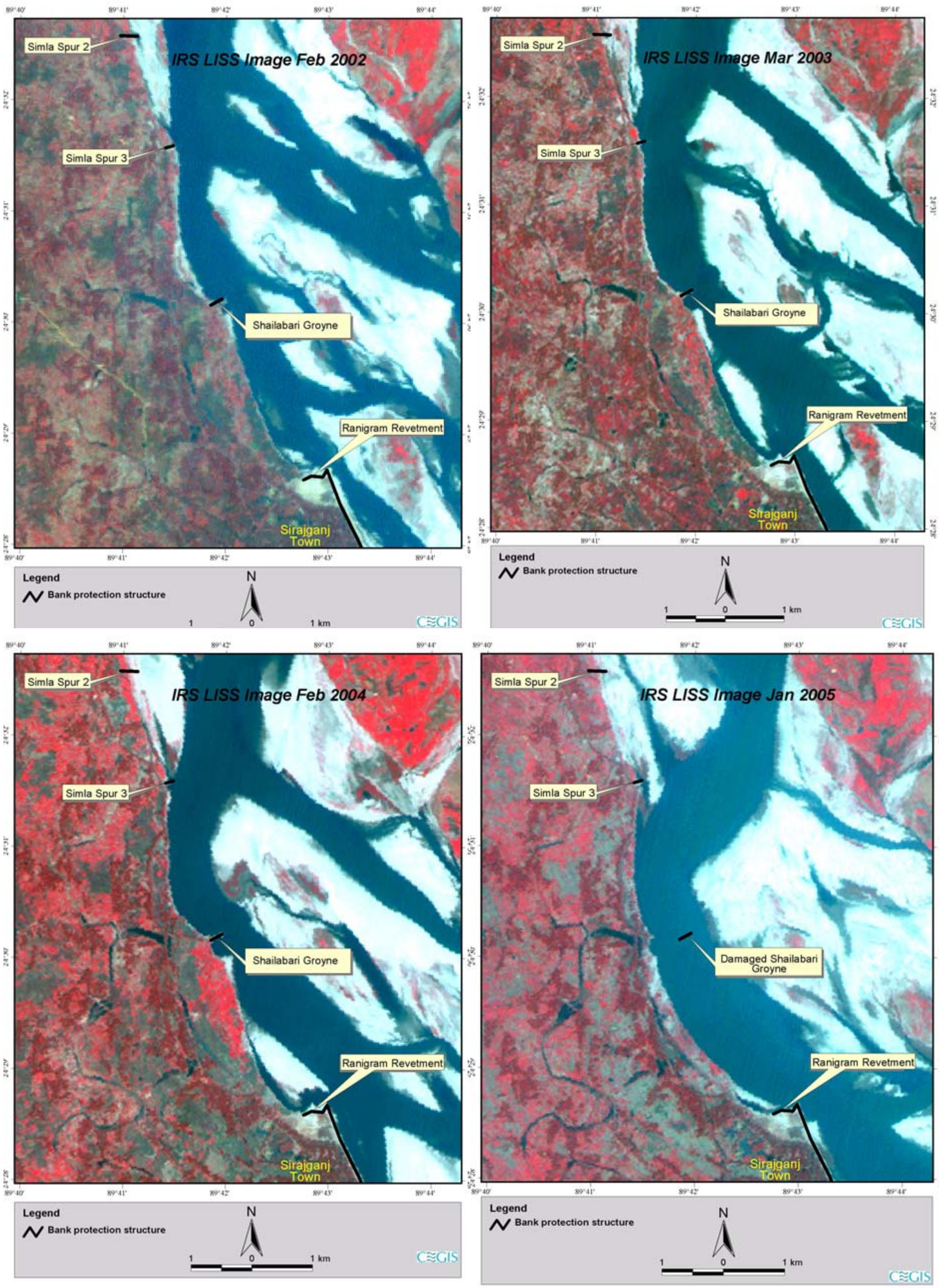


Figure A-30: Channel development at Sailabari Groyne

A.4.3 Concrete Spurs with Earthen Shank

Spurs with concrete head were constructed in Jamuna, Ganges, Teesta, Dharla and Dudhkumar Rivers. Both physical model and mathematical model studies have been carried out before implementing many of these structures. A total of 44 spurs have been built. Among these, 18 spurs in the Jamuna, 9 in the Ganges, 14 in the Teesta, and 3 in the Dudhkumar River have been implemented. Present inventory (Table A.3) shows that in total 8 spurs in Jamuna, 5 spurs in Ganges, 13 spurs in Teesta and 3 spurs in Dudhkumar are functioning without any major defect. Immediate repair cost needed for spurs at Meghai (Spur 1, 2, 3), Simla (Spur 1,2 and 3), Betil, Enayetpur and Panka-Narayanpur is about BDT 60 crore (US$ 10 million; 1 US$=BDT 60).

All spurs consist of earthen embankment (shank) with upper slope protection and 150 m long concrete head section. Spur-heads are built in the dry as the foundation consists of bored piles. To prevent the vertical wall from being undermined due to significant head differences between upstream and downstream (in the order of 15 to 20 cm at the dike and 5 to 12 cm at the spur head), additional concrete blocks were dumped on the ground level.

In this respect, especially vulnerable is the foundation depth of the closing wall of the built structures. This can be kept from the undermining only by continuous high maintenance, and by building up of the spur head to maximum scour depth by continuous dumping of cc blocks. In this way the cost efficiency is lost.

The performances of RCC spurs at Betil, Enayetpur and Panka-Naarayanpur are reviewed below.

(i) Betil and Enayetpur Spurs

Background

RCC spurs at Betil and Enayetpur were constructed on the right bank of the Jamuna River at km.193.53 and km.196.10 respectively in the Belkuchi Upazila of the Sirajganj district. The sites are located downstream of the Jamuna Bridge. The objective was to protect valuable public and private properties (e.g., a large modern hospital at Enayetpur, handloom industries etc.) along the river bank at the two places from the erosion caused by an active western branch of the Jamuna (Figure A-31 and Figure A-32). The Betil and Enayetpur spurs were constructed in the year 2001-2002. Distance between two spurs is about 2.6 km.

Design and construction

The spurs were constructed based on the experiences of similar structures in the Meghai and Simla. No modeling exercises were carried out for this purpose. The works were started in the dry season and continued in the rising flood. Due to shortage of government fund, contractors completed the works with their own investment, which was paid back by the GoB in the next few years (so called *planned liabilities* basis).

Both the structures have a 150 m long RCC head. The length of earthen shank of Enayetpur spur is 1050 m and that of Betil spur is 800 m. The design discharge was taken 64,400 cumec, which is 70% of the 1 in 50 yrs flood. This discharge was used for scour calculation with a multiplication factor of 2.25 in the Lacey's scour depth equation considering the nose of the spur as a side cliff and walls.

According to original design, upstream side of the earthen shank (slope) above LWL was protected by 40 x 40 x 20 cm cc blocks over a geotextile filter and the toe was protected through dumping of 3.0 m^3 per meter to 6.0 m^3 per meter of cc blocks. Toe of bell mouth of the shank was protected by dumping 6.0 m^3 per meter of cc blocks. The RCC part consists of 140 cast-in-situ piles and concrete vertical-face wall above LWL. The length of each pile was 24 m with a diameter of 500 mm. The lowest level of piles was set at a minimum of 6 m deeper than normal scour depth. The RCC part of the spur is also protected from scour by dumping 60.00 m^3 per meter of cc block at the bell mouth, 60.00 m^3 per meter to 5.00 m^3 per meter at upstream and 60.00 m^3 per meter to 3.00 m^3 per meter at downstream side of RCC part of the spur.

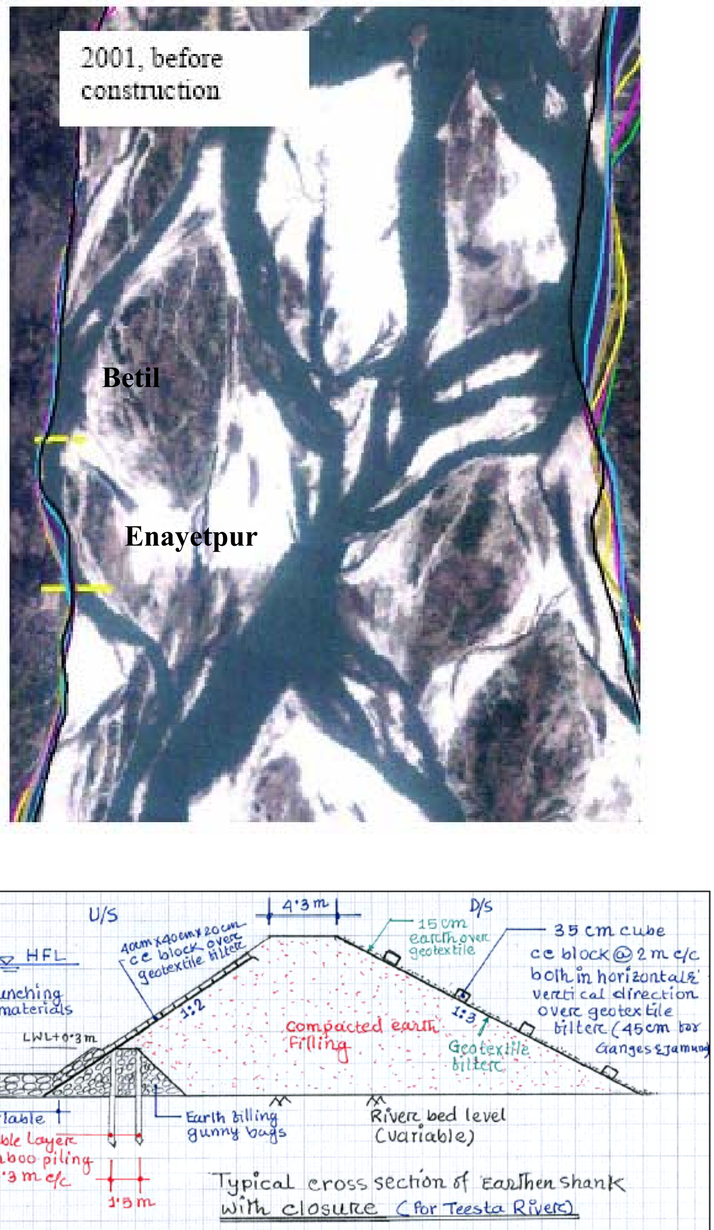


Fig. A.31 Location of Betil and Enayetpur spur and a typical cross-section of earthen shank for channel closure (Jalal 2007).

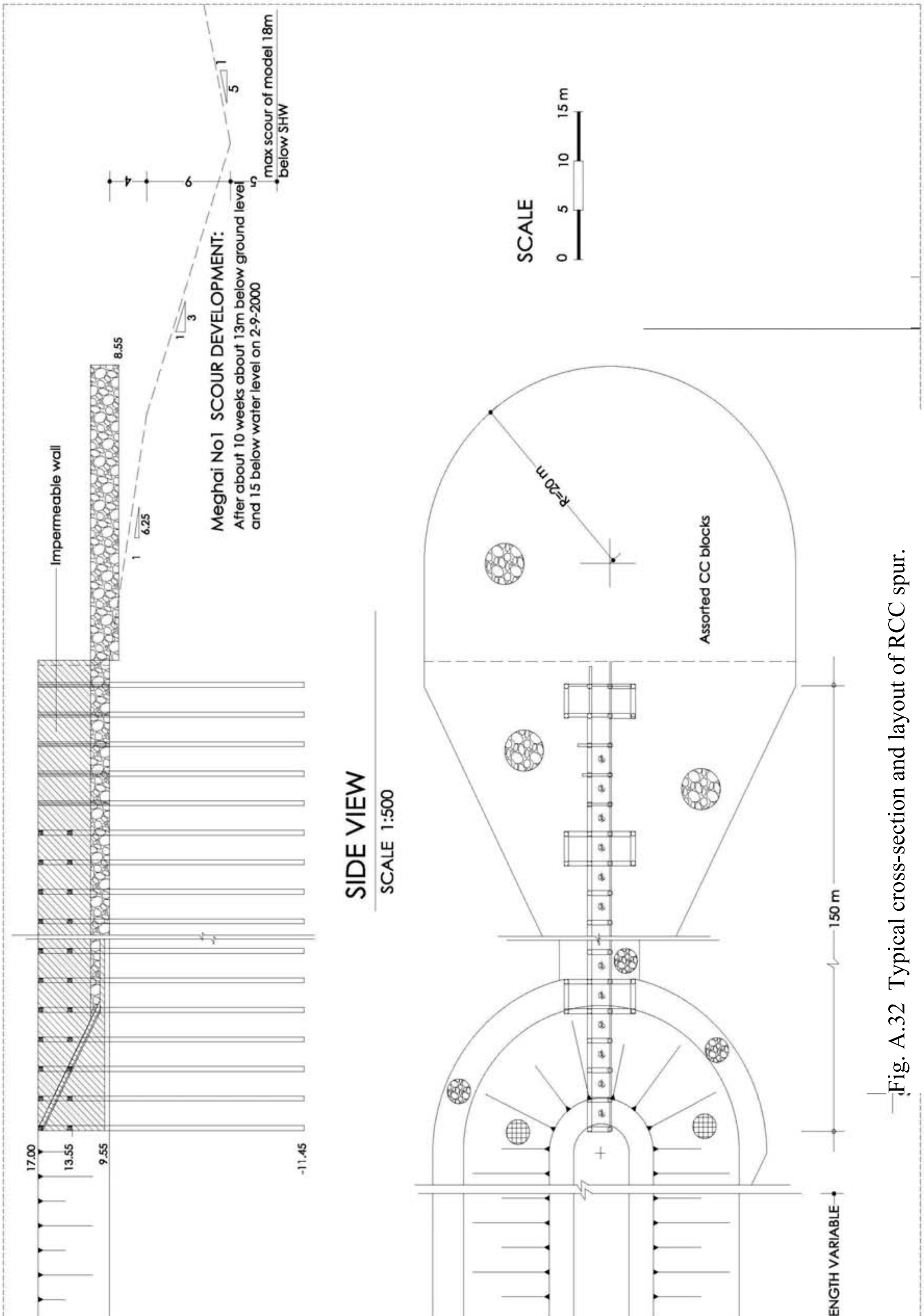


Fig. A.32 Typical cross-section and layout of RCC spur.

Table A.3 RCC Spurs Constructed in Bangladesh.

Name of spur	Name of River and Location & Constrn time	Constrn. Cost (Tk), Crore	Present condition	Remarks/ Repair cost
Vendabari spur-1,2,3	Teesta River, Nilphamari; 1999-2000	2.85, 2.65 and 1.98	Effective	
Shailmari spur-1 & 2	Teesta River, Gangachhara 2001-2002	3.810 and 3.060	Effective	BDT 2 lacs/ year each
Kolkanda spur-1 & 2	Teesta River, Rangpur Under Construction	3.120, 3.590	Not yet completed	BDT 2 lacs / year each
Mahiskhocha spur	Teesta River, Lalmanirhat Under Construction	4.500	Not yet completed	
Mahiskhocha spur-2	Teesta River, Lalmanirhat, 2003-04	4.427	Partly damaged	BDT 95.00 lacs
Rajpur spur 1-5	Teesta River, Lalmanirhat, 2001-05		Spur-4 damaged	
Paikerchara spur-1 Paikerchara spur-2	Dudhkumar river, Kurigram; 1999-2000	2.750 0.480	Effective Effective	
Burirhat spur	Dudhkumar river, Kurigram; 1999-2000	1.500	Effective	
Meghai spur-1, 2 ,3	Jamuna River, Sirajganj, 1999-2000, 2001	19.14, 12.44 & 10.00	Spur-3 Partially damaged	
Singrabari spur-1 Singrabari spur-2	Jamuna River, Sirajganj, 1999-2000	4.680 4.010	Effective Effective	
Subhagacha spur-1 Subhagacha spur-2	Jamuna River, Sirajganj, 2000-2001	4.950 14.520	Damaged Damaged	
Simla spur-1, 2, 3	Jamuna River, Sirajganj, 1999-2000, 2000-2001	3.42, 4.25 & 4.720	Spur-3 Damaged	
Betil spur-1 Enayetpur spur-2	Jamuna River, Sirajganj, 2000-2001, 2001-2002	23.030 19.670	Damaged Damaged	
Hasnapara spur-1 Hasnapara spur-2	Jamuna River, Bogra, 2001-2002	12.630 10.690	Partly damaged Effective	
Baniajan spur	Jamuna River, Sirajganj, 2001-2002	10.810	Effective	
Shashanghat spur	Ganges River, Rajshahi 1999-2000	3.660	Effective	
Panka-Narayanpur spur-1 to 8	Ganges, C.Nawabganj, 2001-2002	9.85, 6.78, 7.0, 7.4, 8.14, 7.99, 7.14 & 7.94	Spur – 1,2,7 & 8 are damaged & ineffective	No new spur, revetments have been built

Damages and maintenance works

Regular monitoring and maintenance of the spurs were not carried out. The bellmouth of the earthen shank of the Betil Spur was damaged in the following monsoon after construction. Some maintenance works were carried out in 2003. During 2003 flood, a slip circle failure occurred in the shank of Enayetpur Spur. The damage was repaired and the bell mouth and RCC part of both the spurs were strengthened through dumping of cc blocks. Again, during 2004 flood, both spurs were severely damaged and the earthen shanks were severely eroded. During 2005, the spurs were inactive because by that time the RCC heads were completely detached from the river bank.

The structures were repaired under Emergency Flood Damage Rehabilitation Project. Reconstruction work started in January 2006 and was completed in July-August 2006. The works continued in the full monsoon under surveillance of the designers and the army. The consultant prepared a revised design for reconstruction of both the spurs. In the design for reconstruction the bell mouth of RCC-part was protected with 50 m^3 per meter of hard material. Both upstream and downstream of the RCC part were protected with 25 m^3 per meter of hard material. Bell mouth of the earthen shank was protected with 50 m^3 per meter of hard material. The upstream toe of the earthen shank from bell mouth protection end towards the bank line was protected with 40 m^3 per meter to 20 m^3 per meter for a length of 220 m. The downstream toe of the earthen shank was protected with 40 m^3 per meter to 20 m^3 per meter of hard material for a length of 150 m.

Costs

For the Enayetpur and Betil spurs, the initial cost in 2000-2001 amounted to BDT 42.00 crore, and the cost of rehabilitation was BDT 25 crore (US$ 4 million) in 2004-2005. The costs of maintenance in 2003 and 2004 were about BDT 9 crore. Total damage repair cost following 5 years after construction was 57% and 109% of the initial construction cost of the Betil and Enayetpur spur, respectively.

Remarks

Betil spur failed primarily due to scour and reverse current, which attacked the lee side of RCC-part and eroded the connection between RCC-part and earthen shank. Strong flow under the RCC-part was another reason for failure. Enayetpur spur failed mainly due to strong angular flow that created deep scour hole in front of the shank and caused its breaching. Local morphological activities of the river near Betil and Enayetpur resulted considerable change in the geometry of the channel near the spurs. This means that the conditions for which the design had been done was changed as well. These adverse conditions contributed to the failure of the spurs. For the same reasons the structures are still now vulnerable to damages.

(ii) RCC spurs at Panka-Narayanpur

Background

Panka-Narayanpur is an area under the Sadar Upazila of Nawabganj district which is close to the Indian border and just downstream of the Farakka Barrage. The Ganges after entering Bangladesh through this area turns abruptly to the east direction and eroded the left bank severely in the past years. During the period 1990-2000, about 45 km^2 of area has been eroded by the Ganges. Lateral bank shifting (Figure A-33) through this erosion was about 7 km and loss to the valuable properties was about BDT 150 crore. To arrest this havoc a project was undertaken in 2001-2002 on the basis of mathematical and physical model studies. The project package included 8 concrete head spurs, 2 cross dams and 16.50 km of flood embankment.

Design and construction aspects

The final design was based on the mathematical model studies. The mathematical model study fixed up the location and protrusion of the 8 spurs in the area. According to the modelers, the bank erosion was supposed to be reduced by 93 to 100% and near bank velocity would be reduced by 42 to 96% after implementation of the measures. The physical model study, however, was carried out with only one option as proposed by mathematical modeling keeping every parameter fixed.

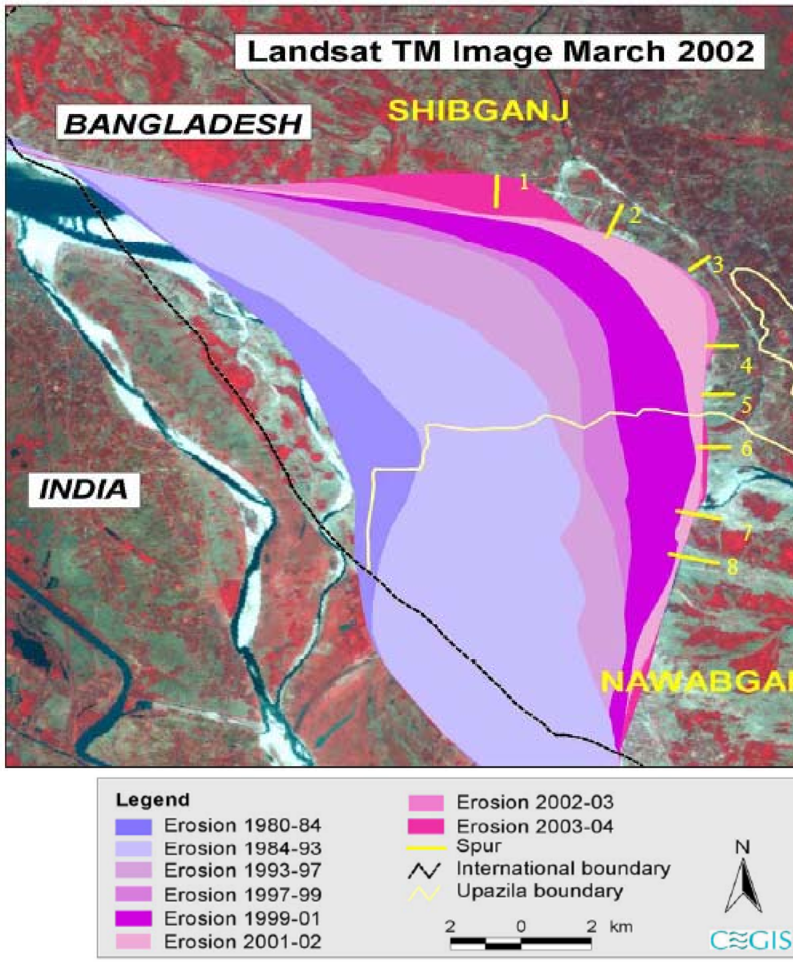


Fig. A.33 Bankline shifting in the Panka-Narayanpur site (CEGIS 2007)

Design discharge for scour calculation was 50,120 cumec (70% of the 25 year flood). The local scour depth was 36 m from HWL with minimum level -12.77 m PWD. Depth of piles was 24 m which is 6 m below the normal scour depth. Launching apron was provided with cc blocks of sizes 450 mm (50%) and 300 mm to 450 mm (50%). At the head of the earthen shank 90 to 105 m^3 per meter materials were used. Two sides of the shank were protected by providing 40 m^3 per meter to 3 m^3 per meter of launching material. Steel sheet pile of length 3.75 m was used below the deck slab to protect seepage from water level difference between two sides. Nominal protection materials were used to protect both sides of the RCC part.

The length of RCC part of each spur was 150 m. The earthen shank length varied, the maximum being 1587 m for Spur No. 1 and minimum 270 m for the Spur No. 3, measured from the embankment.

Damages and maintenance works

In 2002 working season, 7 Spurs (Spur No. 2 to 8) were fully completed but RCC part of Spur-1 was not completed due to high flood and shank failure. In 2003, 66 m RCC part of Spur-1 was completed but during flood season (July) earthen shank of the Spur-1 failed and detached from the concrete head. About 4 km of bank line, both at upstream and downstream of Spur-1 was severely eroded during that time. In late September of the same year, 100 m of the RCC head of Spur-8 was uprooted. Again, in 2004 flood, the main flow being along the left bank of the river, about 3.0 km bank at upstream of Spur-2 was under severe erosion attack. The lateral shifting of bank towards country side through this erosion was 500 m to 1500 m. Due to this embayment, 100 m concrete head of Spur-2 and Spur-7 including some portions of shanks was damaged in August-September 2004. Three Spurs, Spur-2, Spur-7 and Spur-8 whose shanks and concrete heads were damaged in 2003-2004 flood, were completely destroyed by 2005 flood.

To protect the Spur field and their intended effectiveness, hard materials and sand filled geo-bags were dumped in the affected areas. In addition to that, to arrest further embayment of the river 900 m of bank revetment between Spur-3 and Spur-4 and 1080 m bank revetment works at both upstream and downstream of eroded Spur-2 were taken up. The work also failed during the flood of following monsoon.

Recent observation on the groyne field reveals that out of 8 spurs, already 4 are completely damaged. Spur-4 and Spur-6 are under major repair or rehabilitation. To save the spur field and the infrastructure from further erosion a project proposal for protection of the left bank of the river Ganges from the earlier position of Spur-2 to Indo-Bangladesh border for a length of 8.0 km is under preparation.

Cost

The estimated cost of the project was BDT 74.83 crore. The revised (2nd revised) project cost was proposed to be about BDT 140 crore.

Remarks

The failure of the spurs, the extent of erosion at upstream of the spur field and also between the spurs depicts that all these hazards are mainly due to morphological changes of the river. However, lack of coordination between the implementing agencies and unavailability of fund from the controlling ministry in proper time may be another reason for which the structure might have failed. Stopping embayment at upstream of the spur field could have helped to make the spurs more sustainable during flood period. The RCC heads failed due to high velocity of flow occurring through the openings below the spur-wall and subsequent scouring at the base of the piles. Similar phenomena were also observed in the Jamuna. Another important point is that the adopted design and layout of the spurs in this case on about 90 degree bend was not appropriate. Especially the first spur and the following spur, which intercepts the unperturbed flow, should have been designed carefully to guide the flow smoothly around the bend. This also indicates the lack of understanding about meandering channel flow processes.

A.5 General Remarks

The rivers in Bangladesh are morphologically very active. The rate and year-round fluctuation of water flow as well as sediment transport are very high. Consequently, erosion-deposition, channel shifting and bar (char) development are common phenomena in large rivers. Bank materials of these river reaches are mainly unconsolidated, fine non-cohesive and uniformly graded. This type of soil possesses very little resistance against erosive forces generated by the flow of the rivers. It is estimated that more than a thousand kilometer of banklines are under active erosion hazard. Compared to the widespread erosion problem, which is a type of local disaster for the people living along the river reaches, no long tradition of planned bank protection practice exists in the country.

The failure and damage of bank protection and river training works are very common in large rivers. The relevant factors are: (a) Improper construction method, inappropriate time of construction or use of unsuitable materials, (b) Improper design or adaptation after damages due to lack of understanding of reach-scale morphological behavior of rivers, hydrodynamic forces and soil properties, (c) Lack of regular monitoring of the structures and timely repair, maintenance and adaptation, and (d) Unavailability of funds in time and in required amount. Non-attendance of a new erosion affected area, upstream or downstream of a protected reach, is also another important reasons for the failure of some of the protection measures.

The *groynes or spurs* constructed in medium and minor rivers having predominantly single channel are functioning satisfactorily even without any appreciable maintenance cost. The maintenance of similar structures constructed in braided and meandering rivers like Brahmaputra-Jamuna and in some parts of the Ganges has become difficult and very costly though the implementation cost is much cheaper with respect to that of revetment. Although possibility of *outflanking* and damages exist both for groynes and revetments, due to longer protrusion into the main river, the groynes are more vulnerable to such morphological changes.

Failure of groyne structures can generally be referred to scour processes and outflanking of the river stream that may cause severe erosion at the structure. This phenomenon can be observed at nearly every groyne structure along the Jamuna River and other major alluvial rivers in Bangladesh. As a result of changes in the main river stream, the initial design is most of the time not valid after the next flood. Due to complete planform change a rehabilitation or reconstruction of the same structure may not serve the designed purpose at all. The latter can be found in "The life of Sailabari Groyne", which is the most outstanding example for this occurrence

The main reasons for recent failures of RCC spurs in the Jamuna and Ganges are parallel flow along the under-designed earthen shank and undermining or underflow at the middle part of the RCC head. Although the concept of the RCC spurs have been adopted to avoid damages of similar earthen-core structures, the design of the implemented RCC structures have not been taken into consideration of expected unfavorable loading conditions.

The embayment at upstream end of Sirajganj, Kalitola and Ghutail hard points demonstrate clearly that for an efficient and sustainable bank protection, long reaches of bank (several kilometers both upstream and down stream of current erosion site) has to be protected. This provides an additional safe-guard against outflanking. *Protection of a long reach* has the benefit of bank protection, guiding the river to overall stability and also maintaining a navigation channel along the protected reach.

In general, a multitude of constraints are responsible for the development of sustainable river bank protection in Bangladesh. These are (i) inflexible budget allocation and lengthy approval process, (ii) abrupt and sharp changes of river behavior and morphology, (iii) lack of proper prediction tools for channel migration and erosion attack, (iv) high cost of protective works in relation to available fund, (v) high repair and maintenance cost during abnormal events like floods, and (vi) availability of cheaper hard materials.

At this moment, revetments using traditional hard materials involve abnormally high initial cost. The initial and subsequent maintenance cost can be reduced substantially if the traditional hard materials e.g., rocks, boulders, cc blocks are replaced by some economic and sustainable material like geo-bag filled with local sand. The experience in Jamuna Meghna River Erosion Mitigation Project (JMREMP) may be helpful.

APPENDIX - B

Design Examples of Selected Bank Protection Works on Major Rivers in Bangladesh

Design Example - 1

Bank Protection, JMREMP

River: Meghna
Location: Thana: Matlab; District: Chandpur
Chainage N 585648 to N 589498

Design Data:

1.Discharge (Q):	130,000.00	m^3/sec	100 yr flood
2. Velocity (u):	2.50	m/sec	
3. High Water Level (HWL):	6.20	m, PWD	
4. Av.Low Water Level (Av.LWL):	0.55	m, PWD	
5. Flood Plain Level (FPL):	4.25 - 5.00	m, PWD	
6. Wind velocity:	20.00	m/sec	
7.Wind Duration:	1.50	hr	
8. Fetch length:	5.0	km	
9. Significant wave height (Hs):	1.25	m	25 yr return period
10. Wave Period:	4.2	secs	
11.Bank slope, above low water (1V:2.5H):	21.80	°	
12. Bank slope below low water (1V:2H):	26.57	°	
13. Depth Factor (h/d): (from field observation)			

Note: (1) *Bank slope is considered from an average of cross-sections taken at the vicinity, about 50% of the sections are steeper than 1V:2H and rest are flatter, (2) a slope stability analysis to assess stable slope on observed soil data is needed to verify the assumed slope.*

Design against velocity and shear:
Pilarczyk equation

$$\boxed{D_n \geq \frac{0.035 \cdot \overline{u}^2}{\Delta m \cdot 2\,g} \cdot \frac{\varphi_{sc}\, K_\tau\, K_h}{K_s\, \Psi_{cr}}}$$

Dn = Nominal thickness of protection unit D [m] = thickness of cc block		
u = average flow velocity =	2.50	m/s
Δm = Relative density of submerged material = $(\rho_s-\rho_w)/\rho_w$ =		[-]
ρ_w = Density of water =	1000	kg/m^3
ρ_{s} = Density of concrete by stone shingles =	2250	kg/m^3
ρ_s = Density of stone boulders/Rocks =	2650	kg/m^3
g = accleration due to gravity =	9.81	m/s^2
φ_{sc} = stability factor (for current) =	0.65	[-]
Ψ_{cr} = critical shear stress parameter (Shields)=		[-]
K_τ = Turbulance factor = 1.5, for mild outer bends of rivers	1.5	
K_h = depth factor = $(h/D_n+1)^{-0.2}$ =		[-]
K_s = Bank normal slope factor = $[1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$;(for specific material)		[-]
α = slope angle of bank structure = 1V:2.5H (above LWL)	21.80	°
α = slope angle of bank structure = 1V:2H (below LWL)	26.57	°
h/Dn =		[-]

h= average water depth (average depth of water at bankfull stage)	7.50	[m]
Dn = Nominal size of protection unit (assumed) =		[m]
k_s = bed roughness coefficient =		[m]

Note: (3)Logarithmic velocity profiles exist for long stretches with constant bed roughness. For most engineering works as slope or bottom protection, non-developed velocity profile is usually present. (Pilarczyk, 2000)
(4) In this case non-developed velocity profile is considerd.

Non-Developed Velocity Profile

(1) CC blocks, concrete with stone aggregate (multi-layer):

K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =				0.521	
D_n = nominal size of protection element (assumed) = (cc block)				0.30	m
h/D_n =				25	
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =				1.25	[-]
α = slope angle of bank structure = 1V:2H (below LWL)				26.57	°
θ = Angle of repose (cc blocks) =				40	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =				0.718	
φ_{sc} = stability factor (application type) =				0.80	[-]
Ψ_{cr} = critical shear stress parameter (Shields) =				0.035	[-]
D_n =	0.222	m =	250 mm	35.16	kg

(2) Rocks:

K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =				0.503	
D_n = nominal size of protection element (assumed) = (cc block)				0.25	m
h/D_n =				30	
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =				1.65	[-]
α = slope angle of bank structure = 1V:2H (below LWL)				26.57	°
θ = Angle of repose (rocks) =				35	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =				0.626	[-]
φ_{sc} = stability factor (application type) =				0.75	[-]
Ψ_{cr} = critical shear stress parameter (Shields) =				0.035	[-]
D_n =	0.175	m =	200 mm		

(3) CC blocks, concrete with stone aggregate, hand placed/single layer

h = depth of water infront of hand placed cc blocks revetment =				3.5	m
K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =				0.558	
D_n = nominal size of protection element (assumed) = (cc block)				0.20	m
h/D_n =				17.5	
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =				1.25	[-]
α = slope angle of bank structure = 1:2 (below DLW)				26.57	°
θ = Angle of repose =				40	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =				0.718	[-]
φ_{sc} = stability factor (application type) =				0.65	[-]
Ψ_{cr} = critical shear stress parameter (Shields)				0.05	[-]
D_n =	0.135	m =	150 mm	7.59	kg

(4) Geo-textile bags (sand filled gaobag)

K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =	0.467	
D_n = nominal size of protection element (assumed) = (A-type bag)	0.17	m

h/D_n =		44	
$\rho_{s\,=}$ Density of sand bag =		1750	kg/m^3
ρw = Density of water =		1000	kg/m^3
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =		0.75	[-]
α = slope angle of bank structure = 1:2 (below DLW)		26.57	°
θ = Angle of repose =		30	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =		0.447	[-]
φ_{sc} = stability factor =		0.50	[-]
Ψ_{cr} = critical shear stress parameter (Shields), gabions		0.05	[-]
D_n =	0.233 m = 250 mm		

Equivalent thickness of bags = $(abc)^{1/3}$

A-type bags = $(930x530x170)^{1/3}$ = *438 mm* (Equivalent block size)

B-type bags = $(740x440x160)^{1/3}$ = *373 mm* (Equivalent block size)

Note: (i) Both type A and Type-B bags fulfill the minimum requirement, (ii). Equivalent block size against sand filled geobag is considered; (iii) Unit wt of field dry sand is considered for calculating initial weight of sand filled geobag.

(5) Grout-filled mattress

h = depth of water infront of hand grout filled mattress =		4.5	m
K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =		0.532	
D_n = nominal thickness of protection element (mattress) =		0.20	m
h/D_n =		22.5	
$\rho_{s\,=}$ Density of grout =		2300	kg/m^3
ρw = Density of water =		1000	kg/m^3
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =		1.30	[-]
α = slope angle of bank structure = 1:2 (below DLW)		26.57	°
θ = Angle of repose =		30	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =		0.447	[-]
φ_{sc} = stability factor =		0.50	[-]
Ψ_{cr} = critical shear stress parameter (Shields)		0.07	[-]
D_n =	0.109 m = 150 mm		

Uzing JMBA equation

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

(Note: the equation is developed for cc blocks)

CC Blocks as protection element:

D_n = Dimension of cube (assumed)		0.3	[m]
v = design velocity [m/s] =		2.5	m/s
S_s = Specific gravity of revetment material =		2.25	
h/d = depth factor =		25.00	
α = slope angle (1:2) =		26.57	°
θ = Anle of repose of revetment material =		40	°
g = accleration due to gravity =		9.81	m/s^2
D_n =	0.114 m ≈ 150 mm		

Note: Logarithmic velocity profile is not fully developed in these cases, so JMBA formula may *not produce the situation occurring in the field.*

Design against wave action:

Pilarczyk equation

$$D_n = \frac{H_s \cdot \xi_z^b}{\Delta_m \cdot \Psi_u \cdot \Phi_{sw} \cdot \cos\alpha}$$

D_n = Revetment material size (single unit size) [m]		m
H_s = significant wave height [m]	1.25	m
Δ_m = Relative density of submerged material = $(\rho_s-\rho_w)/\rho_w$ =		[-]
g = accleration due to gravity [m/s^2]=	9.81	m/s^2
Ψ_u = system specific stability upgrading factor [-] =		[-]
Φ_{sw} = stability factor for wave loads [-] =		[-]
α = slope angle of bank structure = 1:2.5 =	21.80	°
ξ_z = wave similarity parameter [-] = $\tan\alpha.(1.25T_m/H_s$^0.5)	1.88	
T_m = mean wave period [s] =	4.2	secs
b = wave structure inter action coefficient [-] =		

(1) CC blocks with stone aggregate as protection element
(CC blocks, cubical shape, hand placed in single layer)

Ψ_u = system specific stability upgrading factor [-] =		2.00		
Φ_{sw} = stability factor for wave loads [-] =		2.25		
b = wave structure inter action coefficient [-] =		0.67		
Δ_m = Relative density of submerged material =		1.25		
CC blocks hand placed (s/layer): D_n =	0.365	m ≈	375	mm

(2) CC blocks cubical shape, randomly placed multi layer

Ψ_u = system specific stability upgrading factor [-] =		1.45		
Φ_{sw} = stability factor for wave loads [-] =		2.75		
b = wave structure inter action coefficient [-] =		0.50		
Δ_m = Relative density of submerged material =		1.25		
CC blocks random placed (m/layer); D_n =	0.370	m ≈	375	mm

(3) Broken rocks/boulders, randomly placed

Ψ_u = system specific stability upgrading factor [-] =		1.20		
Φ_{sw} = stability factor for wave loads [-] =		2.75		
b = wave structure inter action coefficient [-] =		0.50		
Δ_m = Relative density of submerged material =		1.65		
Broken rocks random placed: D_n =	0.339	m ≈	350	mm

In consideration of velocity and shear, the hand placed single layer blocks are proposed to be 400x400x200 mm size blocks, their performance against imposed load may be observed in model and prototype.

Design of Launching apron:
(Protection element unit size is based on velocity and shear)
Thickness of protection

T = 1.5x Individual size of cc blocks against design velocity =	375	mm
T_1 = 1.5 x Individual size of rocks against design velocity =	300	mm

Design scour depth, at MDIP = (F 20.00 m (FS Report, Halcrow)

Av.LWL = 0.55 m, PWD
Design Scour level (as per FS) = 0.55 - 20.0 = -19.45 m, PWD
Deepest observed scour level at the vicinity = -12.00 m, PWD
Deepest known scour depth = 12.00 + 0.55 = 12.55 m, below Av.LWL, so design scour depth suggested by the FS Report, being higher than the highest observed scour, is considered for the toe protection; So Design Scour = 20.00 m

Length of launching apron, L = 1.5 x 20.0 = 30.00 m

Considering CC Blocks as protection element;

Inside thickness of apron = 1.5 T = 562.5 mm
Outside thickness of apron = 2.25 t = 843.75 mm

Quantity of hard material (CC block) below Av.LWL (apron material):
V = L(1.5 T+2.25T)/2 = 21.09 m^3/m
Considering 50% extra for underwater dumping = 31.64 m^3/m

Considering Rocks as protection element:

Inside thickness of apron = 1.5 T_1 = 450 mm
Outside thickness of apron =2.25 T_1 = 675.0 mm
Quantity of hard material (rocks) below Av.LWL (apron material):
V = L(1.5 T_1+2.25 $T_{1)}$/2 = 16.88 m^3/m
Considering 50% extra for underwater dumping = 25.31 m^3/m

Considering geo-bags as protection element (mass dumping):

In mass dumping concept, a falling apron is developed from the water line by dumping a calculated quantity of sand filled geobags as a heap below low water level along the river section, the geobags are assumed to launch at a slope of 1V:2H, to cover the slope and future scour holes.

In the Feasibility Study Report protection thickness of 0.61 m on the scour surface for a scour depth upto 17.0 m, and beyond that a thickness of 0.91 m is suggested.

As per FS report, for a scour depth of 20.0 m, volume of material (geobag) requird is:
V = $(20^2+40^2)^{1/2}$ x 0.91 m^3/m = 40.70 m^3/m

The FS indicated a protection unit, each unit comprising geobags of 4 differnet sizes(126 kg, 78 kg, 36 kg and 11 kg), and each size comprising 25% of the unit. Dry weight of each unit is 504 kg and volume is 0.336 m^3.

In MDIP, suggested volume of apron protection for 20.0 m scour depth is 121 units i.e. (121 x 0.336)= 40.66 m^3/m

A comparative statement for protection to Meghna River bank upto a scour depth of 20 m below design low water (+0.55 m, PWD) with different types of materials are:

1. CC Blocks 31.64 m^3/m
2.Rocks/boulders: 25.31 m^3/m
3. Geo-bag 40.70 m^3/m

Note: geo-bag coverage of 0.91 m thick after launching appear to be not realistic, normally average of 1 layer thick coverage, after launching may be achieved. To attain the desired thickness (at least 3 additional layers A-type bag i.e. 0.66 m or 0.61 m as suggested by FS) *sufficient material need to be placed from barges.*

Protection against 20 m scour with a 0.61 m thick launched material (geobag), the volume of geobags required is; $V = (20^2+40^2)^{1/2} \times 0.67 =$ 29.96 m^3/m

Considering geo-bags as protection element (areal coverage):

JMREMP has constructed bank protection by dumping geobags in a regular manner on the existing natural river bank slopes and as wide falling aprons along the toe. Slope protection and falling apron consisting of several layers (minimum 4 layers) of geobags is built from the deep area of the river towards the bank infront of the toe.

Apron length according to suggested formula: 1.5 x 20.0 m = 30 m

In the arial coverage approach followed in JMREMP afterwards, to protect a 20.0 m scour by sand filled geobags with single layer coverage on a slope of 1V:2H ($20 \times \sqrt{5}$ = 45m, slope . length) requires 90 geobags (A-type, 126 kg) i.e. 2 bags/ m^2. The project finally used 4 layers of A-type geobags (4x2 = 8 geobags/m^2) as protection to the river bank.

No of 126 kg geobag per sqm (only A-type bag is used) = 4x2 =	8	nos
Total volume of material placed on the apron (45m wide) =	30.24	m^3/m
Effective size of one A-type geobag is =	0.50	m^2
Volume of geobags (4 layers, A-type) placed/dumped =	0.672	m^3/m^2
Volume of 1 A-type geobag (size: 930x530x170 mm)	0.084	m^3
Average thicknes of A-Type geobag =	0.17	m

Observation by Individual Consultants:
(1).Geobags placed as falling apron below low water level launch down the slope and provide an approximately one bag thick layer of coverage, provided there are enough quantities placed for launching. Observed bank slopes after launching indicates slopes in the order of 1V:2H.
(2). Launching thickness needs modification to (i) achieve the design thickness of protective layer and (ii) provide geotechnically stable slopes in the long run.

Physical Model Study, Vancouver, Canada
1. Single size geobags placed in a revetment were found to be more resistant to hydrodynamic displacement than mixtures with the same maximum size.
2. The 126 kg geobags are stable under high flow velocities. At 4.5 m/s velocities only a few bags were displaced, and even though a number of bags were locally displaced or flipped over, they continue to provide protection.
3. Geobags launch well over sand, but not over other bags unless the slope become very steep (1V:0.8H). This supports the observation of single layer coverage.
4. Thin and wide falling aprons of geobags provide better protection than short thick ones.

Volume and weight of geobags

	weight(kg)	vol (m^3)
Unit	504	0.336
A-type	126	0.084
B-type	78.00	0.052
C-type	36.00	0.024
D-type	11.00	0.007

Size of Geobag after filling sand:
A-type: 930x530x170 mm
B-type: 740x440x160 mm
C-Type and D-Type were not finally used

Protection provided in MDIP

February 2004 to May 2005: (Area Coverage)
Km.44.980 to Km. 46.230:(1250m): Fascine mattress; avg area:30mx1250m; width:30m
A-type:177,300 nos, C-type:306,250 nos

Protection provided per m=177300/1250=	141.84	A-bags	24.65	m^3/m
306250/1250=	245	C-bags		
Falling Apron (5m width);per m =27582/1250=	22.07	A-bags	2.64	m^3/m

(River end of mattress)	41016/1250:	32.81 C-bags

Total protection provided (on Fascine mattress) = 27.30 m^3/m

Km.46.230 to Km.47.030 : (800m): avg.area: 30mx800m; width:30m
A-type:96,000 nos (A-type:4/sqm); Arial coverage;
Protection provided per m length = 120 A-bags 10.08 m^3/m

Km.47.030 to Km.47.630: (600m): Avg. area: 30m x 600m; width:30 m
A-type: 72,000 nos, B-type: 54,000 nos (A-type:4/sqm; B-type:3/sqm)
Protection provided per m of bank length = 120 A-bags, 90 B-bags 14.76 m^3/m

Km.47.630 to Km.48.230: (600m); Avg. area: 30m x 600m; Width = 30 m
A-Type: 72000 nos (A-type bag = 4 per sqm)
Protection provided per m of bank length = 120 A-bags = 10.08 m^3/m
Average protection provided = 11.64 m^3/m (av.)

April 2004 to March 2006 (Arial Coverage)
Km.44.636 to Km.44.746: (110m)

Protection provided per m length =13200/110= (av.20 m width)	120 B-bags	6.24 m^3/m
Falling apron per m of bank line=10560/110= (av.12m width)	96 B-bags	4.99 m^3/m
		11.23 m^3/m

Protection provided at X-sec# 7 (N 588590) in November 2004 show that the materials were placed (arial coverage) from (-) 9.35 m, PWD to (+) 0.75 m, PWD; (6 B-bag per m^2 i.e. 6.11 m^3/m on 20m horizontal distance); -9.35 m PWD is the lowest elevation in the section. Falling apron (8 B-bag per m^2 on 12m horizontal distance at river side end of apron i.e. 4.89 m^3/m) were placed.

Monitoring of the section (N 588590) in August 2006 show that the lowest elevation in the section is in the order of (-)10.0 m, PWD. The steepest under water slope is about 1V:2H; indicating a section stable with a protection of about 11.00 m^3/m (6.11+4.89= 11.00) i.e. a thickness of average 0.34 m, which resembles approximately 2.2 layers of geobag type-B.

In the reach from Km.44.636 to Km.44746; total material dumped under arial coverage and Falling Apron is 11.23 m^3/m, that could protect a scour upto (-) 10.0m, PWD.

Km.46.230 to Km.48.320: (2090 m)

Falling apron per m of bank line =31350/2090=		15 A-bags	6.72 m^3/m
(av. 6m width)	(52250)/2090=	25 B-bags	
(av.10 m width)	167200/2090=	80 B-bags	

Protection provided from Km.46.230 to Km.48.320, from Feb 2004 to March 2006, in two stages (1st level and 2nd level protection):
1st level prot.av. arial cover width is 30 m, and geobag dumped = 11.64 m^3/m (av.)
2nd level prot.av.falling apron width is 16m and geobag dumped= 6.72 m^3/m

Analysis of protection provided (Km.46.230 to Km.48.320) indicates that in total 18.36 m^3/m of geobag on a length of 46 m could maintain the reach in a stable section from February 2004 to March 2006. Average thickness of protection provided is about 0.40 m.

Protection provided at X-sec# 1 (N 586570) in November 2004 show that the materials were placed (arial coverage) from (-) 15.43 m, PWD to (+) 0.75 m, PWD; (120 A-bag/meter on 30 m wide arial coverage i.e. 10.08 m^3/m); -15.43m PWD being the deepest point at the end of arial coverage, falling apron was placed from that elevation towards deeper section/mid river for an width of 16 m, in two stages. 1st stage: 15 A-type bag /m on 6m wide slope distance and

2nd stage: 105 B-type bags on 10 m slope distance i.e.6.61 m^3/m in total.

Monitoring of the section (N 586580) in August 2006 show that the lowest elevation in the section is in the order of (-)17.0 m, PWD. The steepest under water slope is about 1V:2H.

Remarks: (1) The protection works executed in MDIP and monitoring done afterwords show that the coverage of a critical section by about 2 to 3 layers of geobag (i.e. thickness of protection is about 0.34 m to 0.51 m) upto its deep section near the bank (about 45 m from LW line) could provide a sustainable protection to the eroding reaches of the river.
(2). Single size geobags (126 kg) for the River Meghna is preferable than a mix of different sizes.
(3) To arrive at a geotecnically stable slope over the existing bank slope a berm below LWL is a key to upper slope protection (above low water level). The permanent protection shall be built starting on a berm, about 2.0 to 3.0 m wide and at about 1.5 m below Design Low Water (DLW) and subsequently sloping towards floodplain at 1V:2.5H. This measur unload the slope and helps achieving the required safety factor for slope stability;
(4) The underwater berm and slope above that (upto DLW) shall be covered with minimum two layers of randomly placed CC blocks over a layer of geobag (A-type);
(5) The volume of apron protection calculated as per T.S.N Rao is based on "Trench Fill Revetment" method. This may not work in most of the places along the major rivers. The total volume needed for protection/m of bank line (in case of rocks or blocks) may be adjusted with (i) 1.5x thickness of slope pitching from low water line to the the existing bed of the river and (ii) the rest volume of material is placed on the existing bed, (iii) the apron length in that case shall be 1.5 times the anticipated scour from the existing bed.
(6) In case of geotextile bag as protection element 4 layers of geobags shall be placed on slope from low water line to the existing bed level and the apron on horizontal bed shall consist of at least three layers. The apron length in case of major rivers shall be at least 35m.

Design Example - 2

Bank Protection JMREMP

River:	Brahmaputra-Jamuna
Location:	Thana- Bera; District: Pabna
Chainage:	N 654720 to N 655850 and N 656660 to N 657500

Design Data:

1.Discharge (Q):	100,000.00	m^3/sec	100 yr flood
2. Velocity (u):	3.00	m/sec	Observed Maximum
3. High Water Level (HWL):	12.7	m, PWD	
4. Av.Low Water Level (Av.LWL):	3.7	m, PWD	
5 Plood Plain Level (FPL):	8.50 - 9.50	m, PWD	
6 Wind Velocity (v):	20.0	m/sec	
7. Fetch Length:	10.0	km	
8. Duration of wind:	1.75	hour	
9. Significant Wave Height (Hs):	1.0	m	35 yr return period
10. Wave period:	3.4	secs	
11. Design bank slope: 1V:2H :	26.57	°	
12. Depth factor (h/d): (from Observation)		(-)	

Note: (1) Bank slope is considered from an average of cross-sections taken at the vicinity, about 50% of the sections are steeper than 1V:2H and rest are flatter, (2) a slope stability analysis to assess stable slope on observed soil data is needed to verify the assumed slope.

Design against velocity and shear:
Pilarczyk equation

$$\boxed{D_n \geq \frac{0.035 \cdot \overline{u}^2}{\Delta m \cdot 2\,g} \cdot \frac{\varphi_{sc} K_\tau K_h}{K_s \Psi_{cr}}}$$

D_n = Nominal thickness of protection unit (D) = thickness of cc block =		[m]
u = average flow velocity (Observed maximum) =	3.0	m/s
Δm = Relative density of submerged material = (ρs-ρw)/ρw =		[-]
ρ_w = Density of water =	1000	kg/m3
ρ_s = Density of concrete by stone shingles =	2250	kg/m3
ρ_{s1} = Density of stone boulders/Rocks =	2650	kg/m3
g = accleration due to gravity =	9.81	m/s2
φ_{sc} = stability factor,(for current)		[-]
Ψ_{cr} = critical shear stress parameter,(Shields) =		[-]
K_τ = Turbulance factor = (for mild outer bends of rivers)	1.5	[-]
K_h = depth factor (for non-developed/developed vel. profile)		[-]
K_s = Bank normal slope factor = $[1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$		[-]
α = slope angle of bank structure = 1:2.5 (above DLW)		°
h/Dn =		[-]
h = depth of water (average depth of water at toe, bankful stage)	9.0	[m]
d = D_n = nominal size of protection element (assumed) =		[m]
k_s = bed roughness coefficient =		[m]

Note: (3) Logarithmic velocity profiles exist for long stretches with constant bed roughness For most engineering works as slope or bottom protection, non-developed velocity profile *is usually present. (Pilarczyk, 2000)*

(4) In this case non-developed velocity profile is considerd, case with logarithmic velocity is calculated for comparison purpose.

For non-developed velocity profile:

(1) CC blocks, concrete with stone aggregate (multi-layer):

K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =		0.503	[-]
D_n = nominal size of protection element (assumed) = (cc block)		0.30	m
h/D_n =		30	[-]
$\Delta m = (\rho s-\rho w)/\rho w$ =		1.25	[-]
α = slope angle of bank structure = 1:2 (under water slope)		26.57	°
θ = Angle of repose =		40	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =		0.718	[-]
φ_{sc} = stability factor (cc blocks, multi-layer, random placed) =		0.80	[-]
Ψ_{cr} = critical shear stress parameter (Shields),		0.035	[-]
D_n = 0.308 m =	350 mm	96.47	kg

(2) Rocks:

K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =		0.486	[-]
D_n = nominal size of protection element (assumed) = (rocks)		0.25	m
h/D_n =		36	[-]
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =		1.65	[-]
α = slope angle of bank structure = 1:2 (under water slope)		26.57	°
θ = Angle of repose =		35	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =		0.626	[-]
φ_{sc} = stability factor (rocks) =		0.75	[-]
Ψ_{cr} = critical shear stress parameter (Shields), rocks		0.035	[-]
D_n = 0.243 m =	250 mm		

(3) CC blocks, concrete with stone aggregate, hand placed/single layer

h = Depth of water (average) in front of hand placed revetment =		3.5	[m]
K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =		0.558	
D_n = nominal size of protection element (block-hand placed)=		0.20	[m]
h/D_n =		17.5	
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =		1.25	[-]
α = slope angle of bank structure = 1:2.5 (above DLW)		21.80	°
θ = Angle of repose =		40	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =		0.816	[-]
φ_{sc} = stability factor (cc single layer) =		0.65	[-]
Ψ_{cr} = critical shear stress parameter (Shields)		0.05	[-]
D_n = 0.171 m =	200 mm	18.00	kg

(4) Sand-bags (Geo-textile bags)

h = depth of water (average depth of water at toe, bankful stage)		9.0	[m]
K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =		0.450	
D_n = Nominal size of protection element (geobag, A-type)		0.17	[m]
h/D_n =		53	
$\rho_{s\,=}$ Density of sand bag =		1750	kg/m^3
ρw = Density of water =		1000	kg/m^3
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =		0.75	[-]

α = slope angle of bank structure = 1:2 (below DLW)	26.57	°
θ = Angle of repose =	30	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =	0.447	[-]
φ_{sc} = stability factor =	0.50	[-]
Ψ_{cr} = critical shear stress parameter (Shields), gabions	0.05	[-]

D_n = 0.323 m = 350 mm

Equivalent thickness of bags = $(abc)^{1/3}$

A-type bags = $(930x530x170)^{1/3}$ =	*438 mm*	(Equivalent block)
B-type bags = $(740x440x160)^{1/3}$ =	*373 mm*	(Equivalent block)

Note: (i) Both type A and Type-B bags fulfill the minimum requirement, (ii). Equivalent block size against sand filled geobag is considered; (iii) Unit wt of field dry sand is considered for calculating initial weight of sand filled geobag. (iv) moist unit weight of sand filed geobag is considered as final protection element.

Grout-filled mattress

h = depth of water (average depth of water at toe, bankful stage)	7.0	[m]
K_h = depth factor (for non-developed vel. profile) = $(h/D_n+1)^{-0.2}$ =	0.488	
Dn = Nominal size of protection element (mattress)= d	0.2	[m]
h/Dn =	35	
$\rho_{s\,=}$ Density of grout =	2300	kg/m^3
ρw = Density of water =	1000	kg/m^3
$\Delta m = (\rho_s-\rho_w)/\rho_w$ =	1.30	[-]
α = slope angle of bank structure = 1:2 (below DLW)	26.57	°
θ = Angle of repose =	30	°
$K_s = [1-(\sin\alpha/\sin\varepsilon_s)^2]^{1/2}$ =	0.447	[-]
φ_{sc} = stability factor =	0.50	[-]
Ψ_{cr} = critical shear stress parameter (Shields)	0.07	[-]

D_n = 0.144 m = 150 mm

For a fully developed velocity profile:

(1) CC blocks (randomly placed, multi layer):

$K_h=2[\log(12h/k_s)+1]^{-2}$ =		0.392	
$k_s = 2D_n$ =		0.6	[m]
D_n =	0.241 m	300	mm

(2) Rocks

$K_h=2[\log(12h/k_s)+1]^{-2}$ =		0.367	
k_s = 2Dn =		0.5	[m]
D_n =	0.183 m	250	mm

(3) CC blocks (hand placed, single layer):

$K_h=2[\log(12h/k_s)+1]^{-2}$ =		0.181	
$k_s = 0.1D_n$ =		0.02	
D_n =	0.056 m	150	mm

In consideration of current and shear, individual size of protection element:

1. Pilarczyk Formula:(non-developed velocity profile)

(a). CC blocks with stone aggregate (multi/layer):	0.308 m
(b). Rocks:	0.243 m
©.CC blocks, hand placed (s/layer)	0.171 m
(d) Sand filled geo-bags (thickness)	0.323 m

(e) Grout filled mattress (thickness)	0.144 m

2.Pilaeczyk (Fully developed velocity profile)

(a). CC blocks with stone aggregate (m/layer):	0.241	m
(b). Rocks	0.183	m
© CC blocks hand placed	0.056	m

Note:(a) Comparison of protection element size for (i) non-developed velocity profile with (ii) fully developed logarithmic profile show that the cc block (multi layer) in case (i) is about 28.2% higher than that for case (ii), again the size of rocks (multi layer) in case of (i) is about 32.3% higher than that for case (ii).
(b) comparison in case of hand placed cc block appears to be unrealistic.
© Relevant data for different protection element were used for comparison.

Design against wave action:

Pilarczyk equation

$$\boxed{D_n = \frac{H_s \cdot \xi_z^{b}}{\Delta_m \cdot \Psi_u \cdot \Phi_{sw} \cdot \cos\alpha}}$$

D_n = Revetment material size (single unit size)		[m]
H_s = significant wave height	1.00	m
Δ_m = Relative density of submerged material = $(\rho_s-\rho_w)/\rho_w$ =		[-]
g = accleration due to gravity =	9.81	m/s2
Ψ_u = system specific stability upgrading factor =		[-]
Φ_{sw} = stability factor for wave loads =		[-]
α = slope angle of bank structure = 1:2.5 =	21.8	°
ξ_z = wave similarity parameter = tanα.$(1.25T_m/H_s^2)$ =	1.75	[-]
T_m = mean wave period =	3.5	secs
b = wave structure inter action coefficient =		[-]

(1) CC blocks with stone aggregate as protection element
(CC blocks, cubical shape, hand placed in single layer)

Ψ_u = system specific stability upgrading factor =				2.00	[-]
Φ_{sw} = stability factor for wave loads =				2.25	[-]
b = wave structure inter action coefficient =				0.67	[-]
Δ_m = Relative density of submerged material =				1.25	[-]
Size of cc blocks (s/layer):	Dn =	0.279	m ≈	300	mm

(2) (CC blocks cubical shape, randomly placed multi layer)

Ψ_u = system specific stability upgrading factor =				1.45	[-]
Φ_{sw} = stability factor for wave loads=				2.75	[-]
b = wave structure inter action coefficient =				0.50	[-]
Δ_m = Relative density of submerged material =				1.25	[-]
Blocks random placed (m/layer);	Dn =	0.286	m ≈	300	mm

(3) (Broken rocks/boulders, randomly placed)

Ψ_u = system specific stability upgrading factor =				1.20	[-]
Φ_{sw} = stability factor for wave loads =				2.75	[-]
b = wave structure inter action coefficient =				0.50	[-]
Δ_m = Relative density of submerged material =				1.65	[-]
Rocks random placed (m/layer):	Dn =	0.262	m ≈	275	mm

Note: (i) In consideration of velocity, shear and wind generated waves, protection elements

above DLW are proposed as single layer hand placed CC blocks, (size: 400x400x200 mm) on geotextile filter, their performance against imposed load may be observed in model and prototype application.
(ii) Weight of a 300 mm cube CC block is 61 kg and that of a 400x400x200 mm cc bock is 72 kg.

Design of Launching apron:
(Size of Protection element unit is based on velocity and shear)

Thickness of protection
T= 1.5xsize of cc blocks against design velocity =1.5xD_n = 525 mm
T_1 = 1.5 x size of rocks against design velocity = 1.5xD_n = 375 mm

Design scour depth, below LWL (DS) =	30.00	m	(FS Report, Halcrow)
Av.LWL =	3.70	m, PWD	
Design Scour level, DSL=(LWL-DS) =	-26.30	m, PWD	
Deepest observed scour level at vicinity =	-22.00	m, PWD	

Deepest known scour depth = 22.00 + 3.70 = 25.70 m, below Av.LWL, so scour depth (30.0 m) suggested in the FS Report, being higher than the highest observed scour, is considered for the toe protection.
Length of launching apron, L = 1.5xD_S = 45.00 m

Considering CC Blocks as protection element;
(Proposing falling apron from LW to Deeper area of the river)
According to T.S.N.Rao
Inside thickness of apron = 1.5xT = 787.5 mm
Outside thickness of apron =2.25xT= 1181.25 mm

Quantity of hard material (CC block) below Av.LWL (apron material): without void;
V = L(1.5 T+2.25T)/2 = 44.30 m^3/m (without void)
Considering 50% extra for under water dumping = 66.45 m^3/m

Considering Rocks/Boulders as protection element:
According to T.S.N.Rao:
Inside thickness of apron = 1.5 T_1 = 562.5 mm
Outside thickness of apron =2.25 T_1= 843.8 mm
Quantity of hard material (rocks) below Av.LWL (apron material); without void:
V = L(1.5 T_1+2.25T_1)/2 = 31.64 m^3/m
Considering 50% extra for under water dumping = 47.46 m^3/m

Considering geo-bags as protection element (mass dumping):

In mass dumping concept, a falling apron is developed from the water line by dumping a calculated quantity of sand filled geobags as a heap below low water line along the river section, the geobags are assumed to launch at a slope of 1V:2H, to cover the slope and future scour holes.

In the Feasibility Study Report protection thickness of 0.61 m on the scour surface for a scour depth upto 17.0 m, and beyond that a thickness of 0.91 m is suggested.

As per FS report, for a scour depth of 30.0 m, volume of material (geobag) requird is:
V = $(30^2+60^2)^{1/2}$ x 0.91 m^3/m = 61.00 m^3/m

The FS indicated a protection unit, each unit comprising geobags of 4 differnet sizes (126 kg, 78 kg, 36 kg and 11 kg), and each size comprising 25% of one unit. Dry weight of each unit is 504 kg and volume is 0.336 m^3.

In PIRDP, suggested volume of apron protection for 30.0 m scour depth is 181 units i.e. (181*0.336) = 60.82 m^3/m

A comparative statement for protection to Jamuna River bank upto a scour depth of 30m below design low water (+3.70 m, PWD) with different types of materials are:

1. CC Blocks: 66.45 m^3/m (without void)
2. Rocks/boulders: 47.46 m^3/m (Without void)
3. Geo-bag: 61.00 m^3/m (Feasibility Study)

Note: (i) geo-bag coverage of 0.91 m thick after launching appear to be not realistic, as after launching only a single layer coverage may be achieved. To attain a sustainable coverage of at leas three layers, additional materials need to be dumped from barges. to protect the bank slope.

Material (geobag) required to cover a 30 m scour with an average 0.60 m (3 layer of A-type bag) thick cover layer after launching:

$V = \sqrt{5}x30 \times 0.60\ m^3$= 40.25 m^3/m

Considering geo-bags as protection element (areal coverage):

JMREMP is constructing bank protection by dumping geobags in a regular manner on the existing natural river bank slopes and as wide falling apron along the toe. Falling apron consisting of several layers of geobags (minimum 3 layers) is built, as per design, from the deep area of the river towards the bank in front of the toe.

Apron length according to suggested formula: 1.5 x 30.0 m = 45.00 m

To cover a 30.0 m scour by sand filled geobags in total 4 layers of A-type bags are required. Arial coverage is proposed using A type bags from the deeper part of the river bed to cover the bed to low water level covering the designed apron length.

Launch material per m^2 of river bed (horizontal) = 4x2*0.084 m3 = 0.672 m^3
Launch material per m^2 of river bed = $\sqrt{5}$x30x0.65 m = 44.94 m^3

No of 126 kg geobag per sqm (only A-type used) = 4x 2 = 8 nos
Effective size of one A-type geobag is = 0.50 m^2

The observations are:

Individual Consultants:

(1).Geobags placed as falling apron below low water level launch down the slope and provide an approximately one bag thick layer of coverage, provided there are enough quantities placed for launching. Observed bank slopes after launching indicates slopes in the order of 1V:2H; (2). Launching thickness needs modification to (i) achieve the thickness of protective layer and (ii) provide geotechnically stable slopes in the long run.

Physical Model Study, Vancouver, Canada

1. Single size geobags placed in a revetment were found to be more resistant to hydrodynamic displacement than mixtures with the same maximum size.
2. The 126 kg geobags are stable under high flow velocities. At 4.5 m/s velocities only a few bags were displaced, and even though a number of bags were locally displaced or flipped over, they continue to provide protection.
3. Geobags launch well over sand, but not over other bags unless the slope become very steep (1V:0.8H). This supports the observation of single layer coverage.
4. Thin and wide falling aprons of geobags provide better protection than short thick ones,

5 . Quarried rock and boulders have better launching characteristics than bags and concrete

cubes. Geobags were found to launch better than concrete cubes in a comparative test.
6.The tests indicated that shorter more square bags performed better than longer bags

Volume and weight of geobags

	weight(kg)	vol (m^3)
unit	504	0.336
A-type	126	0.084
B-type	78.00	0.052
C-type	36.00	0.024
D-type	11.00	0.007

Size of Geobag after filling sand:

A-type: 930x530x170 mm
B-type: 740x440x160 mm

Protection provided in PIRDIP
February 2004 to May 2005: (Areal Coverage)

N 654400 towards north: (2750m); area coverage; av. width 35m.

Protection provided per m length:	136.74	A-bags	16.82 m^3/m
A:376,046/2750 , B:282,097/2750	102.58	B-bags	
Falling Apron (5m width);per m:	18.18	A-bags	2.24 m^3/m
(River end of AC; A:50,000/2750; B:37,500/2ξ	13.64	B-bags	
(2nd FA; 10m; A:30000/1000):	30.00	B-bags	1.56 m^3/m
(3rd FA; 10m; A:20,000/1000;	20.00	A-bags	4.80 m^3/m
B: 60,000/1000)	60.00	B-bags	
Total protection provided:(AC&FA):			25.42 m^3/m

The work (areal coverage) was done from -20.00 m, PWD to +3.70 m, PWD, on a 45m long (along slope) stretch.

N 656925 towards north: (500 m); Mass dumping;

(i) A-type bags:	30000/500=	60.00	nos	20.16 m^3/m
(ii) B-type bags:	48462/500=	96.92	nos	
(iii) C-type bags:	105000/500=	210.00	nos	
(iv) D-type bags:	343,500/500=	687.00	nos	
Total protection provided:(mass dumping):				20.16 m^3/m

As per design (mass dumping) the material (20.16 m^3/m) was dumped from elevation -5.62m, PWD to +3.70m, PWD.The work was completed in May 2004. In May 2004, the same section after dumping was scoured upto -10.0 m PWD in a slope of app. 1V:2H, After the monsoon the same year the section was silted up to elevation -4.0 m, PWD.

N 657750 to N 661000: area coverage & falling apron: 3280m:

(AC-1) Area coverage (immediate protection);	25.60	A-bags	8.93 m^3/m
(A:57600/2250; B:182380/2250;	81.06	B-bags	
C:240000/2250)	106.67	C-bags	
AC-2:(30m wide including IP)	62.24	A-bags	14.96 m^3/m
(A:204,133/3280;B:613,733/3280)	187.11	B-bags	
Falling apron; 12m wide:	24.00	A-bags	5.76 m^3/m
(A:78,720/3280; B:236,160/3280)	72.00	B-bags	
Total protection provided: (AC and FA):			29.64 m^3/m

A: Design

(1). As per design (areal coverage & emergency dumping) geobags were needed to be dumped from elevation -4.90m to +3.70m, PWD on a slope length of 35m in x-sec # N 659780.51m, N 659430m & N 658329.3 m, located between Nakalia and Mohanganj in Right Bank of Jamuna River.

(2). In x-sec # N 656527 m, coverage through mass dumping was needed from elevation -8.50 m to +3.70 m, PWD.
(3) At the extreme downstream location at x-sec # N 654708 m, geobags needed to be dumped under area coverage from El. -10.50 m to +3.70 m, PWD.

B: Implementation

(Period: November 2005 to April 2006)
(i) At x-sec # N 659780.5 m geobags wre dumped under area coverage from -4.00 m to +3.70 m, PWD on a slope length of 35 m.
(ii) At x-sec # 659430 geobags were dumped under area coverage from El. 0.00 to +3.70 m, PWD.
(iii) At x-sec # N 650329.30 m, geobags were dumped under area coverage from El.-4.75 m to +3.70 m, PWD on 35 m slope length. 12 m falling apron was built afterwords at the riverside end of area coverage.
(iv) At x-sec # N 654708 geobags were dumped under area coverage from El. -10.50m to +3.70m, PWD.

C: Monitoring

Monitoring survey conducted along the same alignment from March 31 to April 3, 2006 did not locate any notable change in the sections.

Observation: (1) The protection work executed in PIRDP show that 25.42 m^3/m to 29.64 m^3/m of geobags were used to protect different critical reaches through arial coverage. The volume of material used is higher than the volume of material required to protect a scour up to 15.0 m (20.46 m^3/m) with a protection thickness of 0.61 m.
(2). Geobags placed in different reach show that the cumulative average thickness of protection provided (arial coverage) at different period varied from 0.56 m to 0.70 m on a apron length of 40 to 45m.

Experience from PIRDP:
(1) The underwater protection shall consist of an arial coverage and a falling apron. The falling apron shall be built in deeper part of the river, at the end of arial coverage, where the bank slope starts to be flatter;
(2)Three layers of bags forms the areal coverage that covers the bank slope from falling apron to low water level;
(3) Only one size of geobag (A-type, 126 kg) is sufficient to attain a dependable coverage;
(4) The thickness of protective layer can be reduced when bags are placed systematically on bank slope and falling apron (minimum 2 layers);
(5) To arrive at a geotecnically stable slope over the existing bank slope a berm below LWL is a key to upper slope protection (above low water level). The permanent protection shall be built starting on a berm, about 2.0 to 3.0 m wide and at about 1.5 m below Design Low Water (DLW) and subsequently sloping towards floodplain at 1V:2.5H. This measur unload the slope and helps achieving the required safety factor for slope stability;
(6) The underwater berm and slope above that (upto DLW) shall be covered with minimum two layers of randomly placed CC blocks over a layer of geobag (A-type);
(7) The slope above DLW shall be protected with one layer of hand placed CC block, stable against design wave load, over a geotextile filter;
(8) A representative drawing accomodating the above experience is shwin in Annex-
(9) The volume of apron protection calculated as per T.S.N Rao is based on "Trench Fill Revetment" method. This may not work in most of the places along the major rivers. The total volume needed for protection/m of bank line (in case of rocks /blocks) may be adjusted with (i) 1.5x thickness of slope pitching from low water line to the the existing bed of the river and (ii) the rest volume of material is placed on the existing bed, (iii) the apron length in that case shall be 1.5 times the anticipated scour from the existing bed.
(10) In case of geotextile bag as protection element 4 layers of geobags shall be placed on slope from low water line to the existing bed level and the apron on horizontal bed shall

consist of at least three layers. The apron length in case of major rivers shall be at least 35m.

Recommendations:

(i) Systematic coverage of the bank slope by minimum threelayers of geobag from the deepest part to the Low Water Line may provide a dependable protection;

(ii) To ensure minimum two layer coverage at least four layers of geobags need to be placed systematically.

(iii) A 15 m wide falling apron at the end of arial coverage (average 45 m wide) towards the deeper section of the river shall be built to attain a sustainable protection. The thickness of falling apron shall be at least 0.50 m or 3 layers of geobag.