Government of the People's Republic of Bangladesh
Bangladesh Water Development Board

4

(4)

River Training Studies of the Brahmaputra River

Master Plan Report

1994

Technical Annexes
Annex 4
Design and Construction

Sir William Halcrow & Partners Ltd.
in association with

Danish Hydraulic Institute
Engineering & Planning Consultants Ltd.
Design Innovations Group

HALCROW

Government of the People's Republic of Bangladesh
Bangladesh Water Development Board

River Training Studies of the Brahmaputra River

Master Plan Report

1994

Technical Annexes
Annex 4
Design and Construction

Sir William Halcrow & Partners Ltd.
in association with

Danish Hydraulic Institute
Engineering & Planning Consultants Ltd.
Design Innovations Group

2

FOREWORD

The BRTS Master Plan Report was issued on 28 June 1993. Comments from BWDB and FPCO were received from July 1993 onwards, and responses to those comments were issued in a single volume on 7 March 1994. The report was approved at the 20th FAP Technical Committee Meeting on 9 August 1994 subject to certain amendments. The amendments have duly been incorporated and the report was reissued in its present form in December 1994.

3

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

MASTER PLAN REPORT

GENERAL CONTENTS

9

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
BANGLADESH WATER DEVELOPMENT BOARD

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

MASTER PLAN REPORT: ANNEX 4 - DESIGN AND CONSTRUCTION

CONTENTS

5

6

TABLES

FIGURES

APPENDICES

Abbreviations used in the Text

BOQ	Bill of Quantities
BIWTA	Bangladesh Inland Water Transport Authority
BRE	Brahmaputra Right Embankment
BRTS	Brahmaputra River Training Study
BWDB	Bangladesh Water Development Board
CPT	Cone Penetration Test
DHI	Danish Hydraulic Institute
EMP	Environmental Management Plan
FAP	Flood Action Plan
FIDIC	Federation International des Ingenieurs-Conseils
ICB	International Competitive Bidding
IDA	International Development Association
JMB	Jamuna Multipurpose Bridge
JMBA	Jamuna Multipurpose Bridge Authority
LCB	Local Competitive Bidding
LWL	Low Water Level
PIANC	Permanent International Association of Navigation Congresses
RE	Resident Engineer
TOR	Terms of Reference

w/9

ANNEX 4 - DESIGN AND CONSTRUCTION

1. INTRODUCTION

1.1 Background

The Brahmaputra - Jamuna River System is the largest of the three major river systems in Bangladesh. An earth embankment, known as the Brahmaputra Right Embankment (BRE) was built during the late 1950s and mid 1960s along the west bank of the Jamuna, extending for some 220 km, as a protection against flooding. On-going bank erosion by the river, however, has led to breaches of the BRE, with attendant crop loss and damage to buildings and infrastructure, and successive costly retirements of the BRE.

The Government of Bangladesh therefore decided to commission physical and mathematical model studies which would provide recommendations for improved performance of the BRE as a flood protection against river erosion.

The Study, which commenced in February 1990, has as its overall object the formulation of a master plan for the long term protection of the BRE. A second object is the design of short term (i.e. priority) measures at critical locations along the right bank for early implementation.

Consistent with the Study Terms of Reference, draft tender documents and drawings for six priority locations were submitted to BWDB in October 1991. Further consideration was given to the layouts of these priority works in the Second Interim Report of December 1991, and in April 1992 the IDA Pre-Appraisal Mission selected, on economic grounds, three locations for implementation. The Consultant was therefore required to prepare revised Tender Drawings, Specification and Bill of Quantities, and to update the Instructions to Bidders and Conditions of Contract in accordance with the latest (December 1991) edition of the World Bank's Sample Bidding Documents.

A further requirement of the Terms of Reference is that the Master Plan Report shall include detailed designs of Phase 1 of the Master Plan, and tender documents. The purpose of this annex is to set out the principles upon which the design of the Phase 1 works is based; it serves the further purpose of outlining methods which a contractor might employ during construction. The design principles of all the Phase 1 works, and the methods of construction, follow closely those already described for the Priority Works in the Design and Construction Management Report (August 1992). The Priority Works constitute Phase 1A of the Master Plan.

The above design considerations and conclusions have resulted in the Drawings and Specifications that form part of the Tender Documents for the Priority Works contracts issued as draft documents in August 1992. The tender documents for the remaining Phase 1 works have been prepared along similar lines, and are presented as Annex 6 to this report.

A further element which is an essential part of overall project planning is to ensure that adequate arrangements are made for the operation and maintenance of the Works. Where a river such as the Brahmaputra is concerned, monitoring will form an integral part of the operation and maintenance procedures. These aspects are considered in Annex 5.

10

Reference should also be made to the Initial Environmental Evaluation (Annex 3) where detailed consideration is given to constructional impacts on the environment, and their mitigation.

1.2 Previous Reports and Technical Notes

As the design concepts have progressed, they have been recorded both in formal reports issued in accordance with the Terms of Reference, and in a series of technical notes and working papers issued both for discussion purposes and to form a permanent record of the status of the design at various stages.

A review of earlier studies and reports is given in summary form in the Inception Report of May 1990. This report also outlined the procedures for data collection, ranking of possible sites and establishing the design parameters.

The First Interim Report of April 1991 reviewed the progress of the Study to that date, including the results of hydrological, hydraulic and geomorphological work, progress of river surveys and mathematical and physical modelling, and a description of the short term works for early implementation.

The issue of the draft tender documents and drawings in October 1991 was accompanied by the Report on Priority Works. The report described in outline the procedure followed for the identification and ranking of the six selected priority locations and set down the values of the key design parameters that were established during the first phase of the physical modelling programme. The report explained how the revetment slopes were determined and the derivation of the design wave characteristics that were used as one criterion for the sizing of the revetment blocks, the other criterion being the maximum near bank velocity. The overall design methodology was presented, with a spread sheet giving the design parameters for the six locations. Extracts from the Report on Priority Works, covering Wave Characteristics, Geology and Tectonics of the Study Area and Ranking of Sites (April 1991), are included herein as Appendices 2.5, 2.6 and 2.8 in that order.

The Second Interim Report of April 1991 described the mathematical and physical modelling components of the Study, reported on the substantial completion of the river surveys and on the geomorphological studies, and set out the basis for the economic, sociological and environmental assessments. The above aspects are covered in detail in the Annexes to the report. The report also included a description of alternatives for the river training works, and listed the principal components of the Master Plan. The river training works layouts proposed in the Second Interim Report led directly to the revised designs reflected in the July 1992 Tender Documents and Drawings for the three locations selected for priority implementation.

A number of technical notes and working papers have been issued as the design has progressed. Amongst the most important of these are:

- Review of Revetment Design, January 1992
- Selection of Geotextile for Bank Slope Protection, February 1992
- Design Note on Revetments for Priority Works, April 1992

These led up to, and are reproduced in, the Design and Construction Management Report of August 1992 as Appendices 2.1, 2.2 and 2.3.

The Final Report (1994) describes how the various elements of the Study have been carried out, and records how the conclusions reached have contributed to the formulation of the Master Plan. The principal design considerations are summarized, with details of how the various parts of the Study have been drawn upon to provide design parameters and guidance as to structural performance. The Final Report provides an important background to the subject matter of this annex.

A further technical note, "Note on Design", was issued to BWDB in March 1993, to accompany detailed design computer print outs for the Priority Works.

The physical and mathematical model studies, and the conclusions thereof, are reported in the four volume Report on Model Studies of March 1993. Detailed descriptions of the physical modelling of four bathymetries (Kazipur, Sirajganj, Fulcharighat and Sariakandi) and the studies of revetment structures are given in the Final Report on Physical Model Studies, prepared jointly by the BRTS team and staff from the River Research Institute, Faridpur, and issued in January 1993.

1.3 Brief Outline of Master Plan Concepts

The objects, scope and assessment methodology for the Master Plan have been presented in Chapter 5 of the First Interim Report. For convenience, the Terms of Reference are summarized below.

The Study Terms of Reference require that the Consultant shall propose and/or work out:

(a) possible measures and their technical and economic feasibility for an improved performance of the BRE for flood alleviation in terms of its overall design standard including its crest level.

(b) River training schemes suitable for the permanent protection of the BRE, including the effects to be expected on the environment; operation and maintenance aspects shall also be taken into consideration.

(c) An implementation schedule for physical/other measures proposed under (b) above; the schedule shall be flexible and shall allow for the phased implementation of permanent river works.

They go on to specify the scope of the Master Plan in that its purpose is the containment of the Brahmaputra river from Chilmari down to the confluence with the Hurasagar. The BWDB have clarified that the scope of the study is limited to that part of the BRE extending from the Teesta confluence to the Hurasagar confluence.

There are certain specific issues to be addressed, including:

- possible effect of the proposed multi-purpose bridge;

- the permanent locations for ferry terminals and crossings;
- whether any proposed fixing of the particular points or stretches along the river will have any detrimental effect elsewhere.

An important requirement is that the Master Plan shall be consistent with the overall strategy of the Flood Action Plan.

There is also the rider that the Master Plan shall address possible alternative measures to river training works - such as embankment retirement, or combination of both types of measure - for ensuring the BRE's performance under possible FAP developments. The selection of the recommended alternative shall be based on technical and economic analyses, and social and environmental considerations.

The BWDB have emphasized that the national policy is to plan for stabilizing the course of the Brahmaputra but that it is accepted that because this is a long term goal involving large investment, it may be necessary to accept local retirement of the BRE as an interim measure.

2. THE PRIORITY LOCATIONS

2.1 Ranking of Priority Locations

The study terms of reference refer to short term works in the context of works that are to be implemented on a priority basis, as distinct from those that will constitute principal elements of the longer term master plan. To avoid the possible misinterpretation of "short term" to mean temporary, the term that has been adopted to describe these bank stabilization works is "Phase 1" works, and the sites where they will be constructed are referred to as priority locations.

The object of the "short term measures" as defined in the ToR is "to implement an immediate rehabilitation of critical sections of the BRE, priority areas shall be identified on the basis of the threat posed by river erosion...... Population centres, important infrastructure, agricultural losses, costs of remedial works and scope of various alternative measures shall be the criteria for the selection of the sites and the type of measures to be implemented immediately. The short term rehabilitation and/or construction of river works should be taken account of in the Master Plan for development of improved flood protection along the Brahmaputra."

The need for an early ranking was dictated by the timescale for implementation. The IDA (World Bank) had indicated their interest in principle in financing a project comprising bank stabilization and river training works on the Meghna and Brahmaputra rivers. The programme for preparation of this Flood Protection-1 Project (subsequently renamed the River Bank Protection Project) included an Appraisal Mission scheduled for January 1992. Preliminary detailed designs for works at not more than six locations on the Brahmaputra were required for consideration by the mission. Before design work could begin, the first phase of the physical modelling had to be completed and site investigations and topographical and bathymetric surveys undertaken.

2.1.1 Initial Ranking: December 1990

In December 1990 a discussion paper entitled "Selection of Locations for Priority Works" was prepared. The approach followed was broadly as outlined in the Inception Report but with a greater emphasis placed on evaluating the consequences of bank erosion and flood embankment breaches and an attempt to relate these to probability of occurrence, based on the scanty and sometimes inconsistent information available.

Two important considerations, which are of fundamental importance both with regard to the ranking and the approach to the definition and design of the works, are:

(a) the rate of erosion is so rapid that the situation is continually changing in a largely unpredictable manner. In most cases there is likely to be substantial property damage during the time that will elapse before initiation of the design process and construction of the works. It has been shown that this period is likely to extend over not less than three monsoon seasons where major river bank stabilization works are involved. This has to be taken into account when evaluating the locations and places practical limits on the methodologies that are appropriate for ranking.

14

(b) In general the option of retiring the embankment to a distance such that there is a low risk of a breach recurring within, say, five years, is strongly opposed by the local community and as such land acquisition becomes a critical constraint. Such action is also widely perceived as contrary to stated national policy in this respect.

For this initial ranking, primary data was collected during a visit to the locations by a team consisting of the BRTS Team Leader, Economist and River Engineer, supported by BWDB field officers. Structured interviews were held at each location with a range of community leaders and local farmers, artisans and businessmen. It was expected that there would be bias in the respondents estimates of the likely consequences of erosion and/or flooding, so the information gathered was carefully scrutinised and the obviously distorted data were rejected.

Throughout the screening of the information considerable attention was given to the establishment of a data set which provided a realistic basis for comparing the relative merits of the alternative sites. All information was assessed in relation to the 1:50,000 and 4 inch to 1 mile topographic maps to check for major inconsistencies with regard to numbers of villages, the areas of land and the major infrastructure said to be affected by breaches in the BRE.

In view of the quality and quantity of the data, it was not considered appropriate to subject the data collected to rigorous quantitative analysis. The appraisal therefore followed an objective multi-criteria approach based on the following principal considerations:

- risk and direct consequence of river bank erosion;
- risk and consequences of flooding from a breach in the BRE;
- capital costs of bank protection works and BRE retirement.

The basis for the quantification of the costs and benefits is set out in the Discussion Paper.

From the wide range of potential selection criteria, the following social and economic criteria were consequently selected for ranking purposes:

- number of people displaced by erosion (social upheaval)
- number of people seriously affected by flooding (social disruption);
- value of land, property and agricultural production lost and/or damaged (economic losses);
- infrastructure at risk for erosion and flooding (economic disruption);
- capital costs of priority works per beneficiary (capital investment);
- Economic Benefit: Cost Ratio (cost effectiveness)

Various rankings were then undertaken on the basis of these different criteria. Finally, different weights were applied to the various criteria in order to derive a set of comparative rankings. Various different weighting combinations were applied to reflect bias towards particular criteria, namely:

- equal weighting (no bias)
- social benefit weighting (bias towards benefit criteria);
- cost effectiveness weighting (bias towards economic justification of capital investment);
- economic benefit weighting (bias towards economic value lost or damaged);
- social and economic benefit weighting (bias towards social and economic benefit criteria)

This exercise demonstrated that the ranking was relatively insensitive to the choice of criteria and to the weightings. The larger townships and administrative centres stand out as the most important category irrespective of the criteria and weightings used.

The next most important category is that comprising the smaller centres of population, which are typically associated with markets or, in the case of Betil, an important handloom centre. Where only agricultural land will be directly protected by the river bank stabilization works then the justification for the investment has to be borne almost entirely by the reduction in risk of a breach in the BRE. This is the situation at those locations where a village has already been totally destroyed by erosion.

The dominant ranking that emerged from this initial assessment was:

1 Sirajganj
2 Sariakandi
3 Fulcharighat

4 Chandanbaisa
5 Betil
6 Kamarjani

7 Kazipur
8 Mathurapara
9 Sonalibazar/Simla
10 Jalalpur

The locations of these sites are shown in Figure 2.1.

2.1.2 First Amendment to the Ranking

Comments on the methodology and interpretation were received from the World Bank and others which highlighted the sensitivity of specific aspects of the assessments:

16

(a) the relative costs of embankment reconstruction compared with bank stabilization;

(b) the quantification of flood protection benefits;

(c) the importance of issues such as the possibility of the Brahmaputra breaking through to the Bangali river in the vicinity of Sariakandi and the security of the ferry ghats.

Following a review of the ranking in the light of these comments the list was amended as follows:

1	Fulcharighat	(3)
2	Sirajganj	(1)
3	Sariakandi	(2)
4	Chandanbaisa	(4)
5	Kazipur	(7)
6	Betil	(5)
7	Kamarjani	(6)
8	Mathurapara	(8)
9	Sonalibazar/Simla	(9)
10	Jalalpur	(10)

The initial ranking position is shown in parenthesis. The changes were heavily influenced by estimates made at the time of the property and infrastructural damage likely to arise directly from the bank erosion. Lack of information on agricultural benefits and disbenefits tended to lead to what later proved to be conservative estimates in this respect. It will be seen that the locations have all remained in the same three principal groups and only the ranking within each group has changed.

2.1.3 Selection of Sites for Preliminary Detailed Design

Since Kazipur had earlier been selected on the grounds of severity of attack, and the first three stood out clearly from the rest it was agreed that the first four locations for which detailed designs were to be prepared were:

(a) Kazipur
(b) Fulcharighat
(c) Sariakandi
(d) Sirajganj

The selection of the remaining two sites for immediate attention gave increased weighting to the probable consequences of the Brahmaputra breaking through into the Bangali river and this moved Mathurapara up at least into the fifth position on economic grounds alone. The choice for the sixth location then lay between Kamarjani, Chandanbaisa and Betil.

Attached as Appendix A is a review of the ranking of the lower six sites in the shortlist in the light of additional information that was available at that time. This exercise resulted in the selection of the last two locations in the first ranking set of six.

17

(e) Mathurapara
(f) Betil.

2.1.4 Selection of Locations for IDA Investment (Phase 1 Priority Works) and Subsequent Implementation

Draft detailed designs for the six locations and the associated ICB contract documents were submitted for review by the BWDB and World Bank in October 1991.

Meanwhile the master plan study was progressing and generating a much improved appreciation of the river behavioural characteristics and quantification of the risks and consequences of BRE breaches. 1-D model simulations of the inundation patterns relating to breaches in different portions of the embankment highlighted the significance of the reach where the Bangali river runs very close to the Brahmaputra. At the same time the concept of the hard-point as a component in a regional level strategy of bank stabilization took precedence over the earlier more site specific focus.

A full economic and sociological assessment of the six locations was prepared and presented in the BRTS Second Interim Report (December 1991). This showed very clearly that in conventional economic evaluation terms investment priorities for bank stabilization were Sirajganj and the reach of bankline between Sariakandi and Mathurapara, where serious bank erosion was making a breakthrough into the Bangali an increasingly likely probability. While investment at these locations showed respectable internal rates of return, the financial and economic returns to bank protection works at the other three locations were very poor (see Figures 2.2 and 2.3).

It was consequently agreed between the Government and IDA (World Bank) that the Phase 1 Priority Works should comprise bank protection works at Sirajganj and Sariakandi/Mathurapara. In the context of the Master Plan, these have been described as the Phase 1A works. During the first phase of the Master Plan, it is expected that hard-points to stabilize the reaches immediately north of Sirajganj and immediately north of Sariakandi will be required. Accordingly, two locations north of Sirajganj - at Simla and Sailabari Groyne and two locations north of Sariakandi - one east of Naodabaga and one approximately 3 km north of Kalitola Groyne, have been included as Phase 1B. Phase 1C will comprise the remainder of the six locations selected originally, namely Fulcharighat, Kazipur and Betil (see Figures 2.4 to 2.8).

18

3. DESIGN OF THE PHASE 1 WORKS

3.1 Definition of Terms

For the purposes of the BRT Study the following definitions have been followed:

Design Objective: a clear statement of the specific purpose and scope of the works to be designed.

Design Concept: a clear statement of the basic form of the works that will be adopted to meet the Design Objective.

Design Approach (or Methodology): a description of the design principles that will be followed to achieve the design objectives, including the methods to be used for analyzing elements of the works and for predicting service conditions.

Design Criteria: the definition of the service conditions to be adopted for the design and the factors of safety and/or acceptable risk levels to be applied. These must be consistent with the design objective.

Design Procedures: formal procedures to be followed for the preparation of design calculations. Typically these will be based on the relevant Halcrow procedures HCP/3.1, 3.2 and 3.3 as defined in the Halcrow Quality Assurance Manual, with minor amendments to suit local circumstances.

3.2 Design Objectives

The objective of the Phase 1 works is to stabilize the right bank of the Brahmaputra in specific locations where bank retreat is threatening the integrity of the Brahmaputra Right Embankment or where important population centres and infrastructure are being threatened.

As far as possible such works are consistent with the longer term aims of the Brahmaputra Master Plan.

Because of the rapidity with which the river conditions change, the extent of the works cannot be firmly defined in advance of construction and the design must be such that it can be adapted to suit the conditions prevailing at the time of construction.

The design life of the works has been set at 30 years. This period is used when calculating the probability of exceedance of design conditions and therefore for assigning design values to key parameters such as scour depth and near bank velocity. There is therefore a high probability that given adequate and timely preventative maintenance the actual functional life of the works will be considerably in excess of this.

The design of the works must be such that the risk of outflanking due to the development of an upstream embayment or as a consequence of out-of bank-flow is low.

The works are to be designed for ease of maintenance using equipment and plant that is consistent with normal working practices in Bangladesh. The design of the works shall be such that maintenance requirements are low.

3.3 Historical Perspective

On the Brahmaputra, within the study reach, the focus historically has been on the stabilization of the river bank in areas where erosion is resulting in the loss of agricultural land or threatening property and infrastructure. Where the stabilization results in some accretion then this is an added benefit. Although the maintenance of navigation channels requires a regular programme of dredging the stabilization of such channels has not to date been addressed by structural measures, the preferred option being to relocate the ferry terminals and other berthing points when the dredging requirements become excessive. These relocation costs are relatively low and through experience the task can be undertaken with little disruption to the flow of traffic.

A considerable amount of investment has been made in bank stabilization measures over the past 20 years but the only example of relative success is the Sirajganj town protection, consisting primarily of multiple layers of randomly dumped concrete cubes. The protection of Sariakandi by means of the Kalitola Groyne may be considered a partial success in that the erosion was controlled for long enough to see the main point of attack move further downstream. The survival of Kalitola Groyne is probably attributable to the fact that the most severe conditions did not develop at this location before the eroding bend died naturally due to a transfer of anabranch flow. If conditions such as experienced at Fulcharighat or Kazipur had developed it is unlikely that the groyne would have survived.

The inference is that the approach to design and construction currently followed will suffice for moderately aggressive conditions but will not provide the level of performance required under the more severe conditions. In this respect it is significant that the size of protective block required to resist flow drag is proportional to the square of the velocity. Thus for example if an increase in velocity from 3.0 to 3.6 m/s requires an increase in block size from 35 cm to 50 cm (brick aggregate), then a further increment to 4.2 m/s calls for a block size of about 70 cm weighing almost 700 kg. The high velocities that can be induced round the noses of groynes and other protruding hard-points therefore demand larger blocks than can be easily handled by hand.

The inadequate size of armour blocks is not the only potential cause of failure. The fine uniformly graded silty sand, of which the bank and bed is primarily composed, has very low strength and transport resistance. The turbulent flow induced by the armour layer will tend to draw the sand easily through the coarse matrix of dumped blocks; pore pressures in the bank during falling stages will exacerbate this tendency, resulting in rapid slumping of the underlying material and its consequent exposure to the full force of the current as the protective layer subsides. Traditional khoa filters would be ineffective under these conditions, even if they could be placed to specification.

Finally there is the major problem of controlling the distribution of blocks when placed in depths of 15 m or more, with flow velocities even at low flow of more than 1.0 m/s, and high turbidity. Because of the high material cost, there is a natural tendency to place only the absolute minimum of blocks. Under such conditions uneven coverage is almost inevitable, and it requires only one exposed area to initiate a progressive failure of the whole system.

Given these constraints and difficulties, the low success rate for structures built in accordance with the normal practice is only to be expected.

3.4 Design Concept

3.4.1 Short and Long Term Considerations

The study Terms of Reference use the term short-term works to describe those works that are identified as required to treat immediate problems of severe bank erosion. In this context the short-term refers to the timescale for implementation rather than implying a limited life expectancy. For the purpose of this report, "Phase 1" is considered as being synonymous with "short term".

It is self-apparent that, for the conditions encountered on the Brahmaputra, designing bank stabilization works for anything other than to perform under severe conditions cannot be cost-effective. By definition, the locations that are identified as requiring priority attention are those where the hydrodynamic and morphological conditions are in or close to the severe category. For such works to survive for even one season the design must be such that the costs involved will be measured in hundreds of millions of taka, or millions of dollars. With this level of investment, the works can only be justified if they have a life expectancy measured in hundreds of years, and this in turn implies that the works must be designed to survive under the most severe conditions that may reasonably be expected to occur during that period.

The ToR also require that the short term works should wherever possible be consistent with the longer term strategy for river planform stabilization. This reinforces the principle that the short term works must be designed to satisfy long term requirements with regard to durability and performance. Long term in this context is taken to mean a life of 30 years for the concrete blocks, which are the most degradable element.

With regard to hydrodynamic and morphological considerations, the works are all designed to perform satisfactorily under adverse conditions associated with a flow event with a 100 year return period, giving a combined risk of exceedance of not more than 1 percent. Under more severe conditions some displacement of the protective blocks would occur, requiring timely remedial action, but rapid progressive failure should not follow.

The requirement that the short term works should if possible contribute to the longer term strategy is consistent with the concept that the phasing of major river training works should in general follow the priorities set by the severity of the current problems.

3.4.2 Use of Local Resources

The ToR require that where possible and practicable the designs for the short term works shall allow for a maximum use of local labour and materials. Since the routine maintenance will certainly be geared to local labour and material, this requirement has led to the selection of brick aggregate concrete blocks as the preferred principal material for the armour layer for all slope protection exposed to the main river flow.

The large scale manufacture and handling of blocks by labour intensive means is well established and appropriate for the smaller size (55 cm). For the larger sizes (72 and 85 cm) some form of mechanical handling will be required but the casting can remain labour intensive.

Alternative forms of armour material that would be functionally appropriate include boulders, quarried rock, stone aggregate concrete blocks and various forms of flexible matting. Boulders are available from the Sylhet area of Bangladesh but there are only limited quantities of the larger sizes required to satisfy the Brahmaputra conditions and this cannot be considered as a reliable source for this scale of works. Quarried rock is only available from India, Bhutan or further afield and this uncertainty of supply has to be offset against the greater durability potential, and possibly marginally better performance, of rock. Stone aggregate concrete has a higher density than concrete made with brick aggregate, meaning that block dimensions can be reduced and some saving in cement thereby achieved, but this advantage will normally be offset by the higher cost of the stone.

The use of flexible mats has potential cost advantages in a situation such as this where material costs are a major factor but a higher level of sophistication is required, both in terms of plant and skills, for both initial placing and subsequent maintenance. This tends to offset the initial cost benefit. Since the mats rely on interlocking of the elements, any local failure that is not promptly treated can lead to extensive failure of the revetment in a relatively short time. It is possible that this form of armouring might be offered as a cost-saving alternative by an experienced international contractor with the appropriate plant. In which case its suitability under the specific conditions of the project would have to be carefully assessed.

In situations where the forces acting on the revetment are less severe and maintenance is more easily undertaken, for example on the slopes of cross-bars, wire covered brick mattressing will be used. This form of protection makes maximum use of local material and resources but is not suitable for the more arduous conditions to which the main bank revetment is exposed.

The other major elements of the work are the supply and laying of the geotextile filter and the dredging. Both these require heavy plant and, in the case of the geotextile, imported material, which is unavoidable, although the scale of works on this and other projects anticipated in the coming five years that will require geotextiles may encourage international manufacturers to set up facilities in Bangladesh. The raw materials would still have to be imported.

3.4.3 Planned Maintenance

Although the design of the revetment and other works will be in accordance with criteria that are chosen to minimise maintenance, it is anticipated that some local failures will occur that require timely maintenance. It has been assumed that after the first season on exposure to the monsoon flows it will be necessary to place an additional 5 percent of blocks where weaknesses have been exposed. Thereafter the replacement rate should drop to less than 1 or 2 percent per year.

Failure of the slope protection is most likely to be associated with the weakest feature of the structure which is the falling apron. Regular monitoring of the performance of the apron and the slope immediately above it is an essential part of the maintenance programme so that remedial action can be taken before instability develops.

3.4.4 Planform for Bank Stabilization Structures

Introduction

The object in all cases is to stabilize the bankline. The differences lie in the degree to which the stabilization is to be imposed. At the most rigorous, the bankline is to be completely defined and for this purpose some form of revetment is the appropriate treatment; this is usually applicable to a relatively short stretch of bankline (e.g up to 2 km) where some specific object or concentrated area is to be protected, such as at Sirajganj. If the exact bankline configuration is not important but there are fixed limits on the extreme positions that the bankline may adopt relative to the mean line, then some form of intermittent erosion resistant structure is appropriate. This may take the form of groynes or hard-points.

Definitions

In this context groynes are defined as hardened structures that protrude into the main channel, when constructed, with the primary object of deflecting erosive flows away from the bankline. By their nature, they move the centroid of the channel away from the original bankline and thus unless complementary groynes are provided on the opposite bank (or for some other reason it is erosion resistant), erosion of the other bank will take place to an equal extent. The concept that groynes result in a net land gain may therefore be illusory. Groynes are in general relatively costly to build because of the fact that they have to be constructed in deep flowing water and also because their flanks are exposed to high velocity flows during high river stages. Although scour associated with a groyne can be well in excess of that normally found in a river, the deepest point is typically situated more than six times the depth away from the groyne structure and therefore presents no significant threat to its stability. Scour alone is therefore not a major consideration.

Hard-points differ in concept in that there is no attempt to actively deflect the river. The object is to hold the bankline at suitable intervals and to allow it to take up its natural shape in between. Until the system settles down there may be some continuing loss of land but this will be substantially less than would have occurred without the intervention. The spacing between the hard-points will be determined by the maximum depth of embayment that may be permitted between the structures. Hard-points are typically constructed with their river face in line with the existing bank; this simplifies construction and in particular avoids the high cost of placing materials below the water line while exposed to the full river flow (or alternatively the cost of massive coffer dams). After an upstream embayment has formed, the hydraulic conditions at the upstream nose of the hard-point will become much the same as those of a groyne but the exposure to the highest velocity flows will normally be limited to the nose alone, since there is no exposed flank to protect.

In both cases provision must be made to prevent out-of-bank flow from scouring a channel on the land side of the hardened structure which could result in outflanking. This is most simply and inexpensively achieved with an earth embankment, known in Bangladesh as a "cross-bar", with light protection on the slopes against wave action. An alternative in situations where flood plain conveyance may be considered important would be a low level erosion resistant overspill cill linking the hard structure to the BRE. The disadvantage of such an arrangement would be that there would be no land access to the main groyne or hard-point for maintenance during periods of out-of-bank flow, unless the structure took the form of a bridge or multiple culvert, thereby considerably increasing the cost.

Choice Between Groynes and Hard-Points

The choice between groynes and hard-points will depend on three considerations: (a) relative cost per linear metre of stabilized bank line, (b) whether thalweg alignment is in itself a primary consideration and (c) the environmental impact.

It has been noted above that unless complementary groynes are provided on the opposite bank (or for some other reason it is erosion resistant), the construction of one or more groynes on one bank of a river will normally result in erosion of the other bank to an equal extent. The concept that groynes result in a net land gain may therefore be illusory. In this respect, the environmental advantage of a groyne over a hard-point, particularly in terms of social benefit, is therefore limited to situations where the treatment of both banks is appropriate.

3.4.5 Active Channel Training

The excavation of pilot cuts and the construction of temporary submerged sills to encourage a river to follow a particular course are well-established practices on meandering rivers. The concept could have some merit on a smaller braided river but the logistics involved for a river the size of the Brahmaputra put the concept in a different perspective.

Simulation of a typical situation involving an anabranch bifurcation around a char was carried out using the 2-D modelling system (see Part 13 of the BRTS Report on Model Studies for details). A dredged trench sized on the capacity of a medium size cutter-suction dredger, typical of those operated by the BIWTA and BWDB in Bangladesh, was simulated in one branch and the equivalent material was dumped in the other channel. After a simulated period of 80 days both the mound and the trench had extended downstream by 4,000 m (50 m/d) and the flow split altered accordingly.

A further simulation was carried out without any movement of material but altering the angle of attack of the flow approaching the bifurcation by only ten degrees. This resulted in double the rate of accretion and erosion to that of the earlier simulation. The inference is that while the pilot channel excavation was very effective with a symmetrical approach channel, the effect would be almost totally masked by a modest change in the angle of approach due to the shifting of the upstream bend. Since this level of shifting can occur very quickly as the flow increases, the concept does not appear very promising.

These findings have important implications with regard to the training of an anabranch bifurcation.

3.4.6 Stabilization of Nodal Reaches

Inter-island reaches where the anabranches tend to combine into a relatively stable single thalweg channel are a feature of many braided rivers. These are often referred to as nodes, although this term with its implication of a degree of fixity and uniqueness may be misleading.

The Jamuna section of the Brahmaputra has some fairly well developed nodal reaches that have remained in approximately the same location for over one hundred years while others are far less well defined and have tended to wander both longitudinally and laterally. The indications are that the islands are becoming stronger features and as a consequence the nodal reaches are in some cases declining in importance while in others they are becoming more pronounced.

There is a school of thought that favours the stabilization of these nodal reaches as the first step in the stabilization of a braided channel. The principle being that with the nodes stabilized the channel in between has relatively little freedom of movement. The argument is stronger in the case of a river that has a greater tendency to wander than to increase its width, as would typically be the case of an older river than the Jamuna. In the case of the Jamuna where the widening trend appears to be generally stronger than the wandering trend then the case is less clear. With this arrangement there is no direct control over the bank erosion process and even after node stabilization the tendency to widen will continue for some time, because the char formation process will be unaffected.

3.4.7 Bend Stabilization

Bend stabilization is in a sense the complement to cross-over stabilization. With this approach the primary emphasis is on stabilizing the major scale bends that are typically associated with the anabranches on either side of the island zones and which are where the majority of the bank erosion takes place. Thus treating the bends may be seen as a more direct approach to reducing bank erosion in the short-term.

In the longer term node stabilization would still be indicated in order to prevent the river responding to the bend constraint by shifting the node position, thereby distorting the pattern and leaving the bend constraint structures in sub-optimal locations.

The first stage structural works would consist of revetted hard-points on the current bank line and this would be followed by intervening and opposing works to form the planned alignment.

3.4.8 Hard-Point Concept

The hard-point approach is the lowest level of intervention aimed at controlling the planform of the river. No attempt is made to confine the river to one or more defined channels, rather the concept is to stabilize the boundaries of the braid belt (the area swept out by the shifting braid pattern over a period of time). Depending on the magnitude and spread of the problem, one or more short lengths of the bankline are hardened by the construction of conventional bank revetment with upstream and downstream terminations. A lightly protected cross-bar on the flood plain links the hard-point to a set-back flood embankment and prevents out-of-bank flood flows from bypassing and outflanking the hard-point. The length of bank that is hardened at each location is determined by consideration of the likely embayment geometry upstream;

the object being to ensure that there is an acceptably low risk of the embayment threatening the integrity of the cross-bar.

The spacing between hard-points is determined by the depth of embayment that can be accepted and compatibility with the dominant anabranch wavelengths. An important consideration is that they should be sufficiently close together to prevent the development of one of the large and very aggressive bends that are a persistent feature of the river; another is that they fit into a regional level pattern of hard-points designed to stabilize complete reaches of the river. Where possible their locations are selected so that they provide direct protection to infrastructure or other investment in addition to their larger scale function of reach stabilization.

The function of the hard-point is to limit the maximum extent of bank erosion that can occur in the same manner as would occur in nature were the river to encounter a more erosion resistant formation. As such, they can be considered as playing a passive role.

If constructed only on one bank of a river that has a natural tendency to widen to achieve its regime width, hard-points will not prevent this widening process continuing and the river may compensate by increased bank erosion on the opposite bank. However in the case of the Brahmaputra the widening process is closely linked to the creation and development of chars, which in turn is associated with the generation of sediment through bank erosion. Stabilization of one bank may thus be expected to slow down char growth and thus reduce the tendency of the river to widen. Only by further survey and monitoring following the construction of the first phase of hard-points can these relationships be better quantified. At worst it may prove necessary over a period of time to provide matching hard-points on the left bank.

3.5 General Design Considerations

3.5.1 Geotechnical

Geologic and Tectonic Characteristics of the Area

Various geologic and tectonic reports that have been collected for this study confirm the very complex geology of the Bengal Basin, belied by the relative simplicity of the Holocene mapping completed in 1991. Recent work on the seismicity of Bangladesh suggests that many of the faults are still active and that uplift is continuing on either side of the Brahmaputra course in the form of the Barind Tract and Madhupur Forest upthrust blocks. The two tectonic main components of the Bengal Basin are the stable shelf which forms part of the Indian Platform area to the northwest and a subsiding geosyncline to the southeast; the dividing line between the two is known as the Hinge Zone. This zone is reported to pass through Calcutta, Kushtia and Mymensingh and therefore crosses the Brahmaputra obliquely around Sirajganj (see Appendix B).

Subsidence is still occurring in the Bengal Basin south and east of the hinge zone (except the upthrusted Madhupur block), while north-west of the hinge the shelf is relatively more stable. It has been suggested that the bed gradient of the Brahmaputra may on this account be steeper south of the hinge zone than north, although in reality the influence of such deep-seated and relatively slow tectonic movement is likely to be masked by the ability of the river to adjust its bed level relatively rapidly in response to much shorter timescale influences.

26

Soil Characteristics

A considerable amount of geotechnical investigation has been undertaken along the Brahmaputra and in its vicinity for the Jamuna Bridge project and earlier for the East-West Interconnector. This provides a good overall view of the soil characteristics of the area and indicates that the riparian soils are relatively uniform in character. Further site investigations were carried out under the BRTS to confirm the stratification changes and to check the uniformity of the soils. This programme comprised twelve boreholes on the right bank and one on the left bank (see BRTS Report on Priority Works, October 1991, for further details).

The Jamuna Bridge investigation showed the significance of the mica content in the Jamuna sands on deformation characteristics of the sand and interpretation of CPT results. The site investigations and laboratory tests were carried out by international companies to a high standard.

Analyses for the Stage II Jamuna Bridge Studies and for the Brahmaputra Barrage Engineering Appraisal both indicated that soils at Sirajganj and Bahadurabad were prone to liquefaction from design earthquakes to depths of up to 17 m, possibly to 21 m at Sirajganj. This consistency in depth, despite very different design earthquakes arises from denser strata being present on the surface at Bahadurabad than Sirajganj.

The conclusions that emerged are that the soils which influence the Brahmaputra, and any engineering works controlling the Brahmaputra, are primarily micaceous fine to medium silty sands, loose near the surface, generally finest at the surface and becoming coarser with depth.

Outside the limits of the BRTS study, near the Atrai-Gurai river system the alluvial soils are more clayey and the Brahmaputra also impinges on an area on the left bank a short distance upstream of Sirajganj, which is reported to have a higher proportion of alluvial clay. A single borehole sunk in this area under the BRTS investigation programme intercepted a 2 m thick clay layer approximately 3 m below ground surface but the significance of this on the morphology of the river is not obvious.

Analysis of the borehole logs and classification tests leads to the following interpretation:

- Within the depth of interest for bank protection works (ie about 30 m) no trends in stratification can be discerned from north to south adjacent to the right bank of the Jamuna.

- The vertical stratification is quite variable, even between adjacent boreholes sunk at the same priority sites, but no trends can be discerned which should be used for the design of revetments, groynes and embankments. The soils are generally finer near the surface than at depths.

- The soils vary between clays of low plasticity; through silts of low plasticity; non-plastic silts; non-plastic silty fine sands/fine sandy silts; fine sands; and medium-fine (predominantly fine) sands. The sands are extremely uniform, generally with a coefficient of uniformity of between 2 and 3. They are all described as being slightly micaceous.

27

- The predominant soils below a depth of about 10 m are non plastic uniform fine sands with small amounts of medium sand and silt. In the top 10 m the deposits are finer, ranging from clays of low plasticity through silts of low plasticity to non-plastic silts and silty fine sands. At one location silty fine sands were present to a depth of about 20 m and hence their presence to this depth elsewhere cannot be ruled out.

Liquefaction

Loose fine cohesionless deposits may liquefy under non-dynamic loads, depending on their relative density and initial state of stress. Observations in the field, reports of "fluidised" failure of structures in Bangladesh in the past, and observations by Coleman on his visit to Bangladesh in 1990, all point to the possibility that such phenomena may have occurred here and could well occur again in the future. An analysis has therefore been carried out to see whether the present site investigation and data from the Jamuna Bridge geotechnical report might support this hypothesis.

The results of this analysis show that data from both sets of investigations tend to confirm some strata are sufficiently loose and at an initial state of stress from which liquefaction could ensue. The bulk of the data show the soils to be in a condition that is not conducive to liquefaction occurring. This analysis tends to confirm what little evidence there is that liquefactions or partial liquefactions are possible, but not common.

For the conditions prevailing on the right bank of the Brahmaputra the relatively low risk of failure due to liquefaction would not appear to justify the technical difficulties and high cost of densification, much of which would have to be carried out in the saturated sands.

Piping and Erosion

Seepage forces during the falling stage will cause piping in these fine soils unless filters are provided. The most essential area for the filter is where there are waves and this will also cover the areas where seepage forces are greatest during the falling stage of the river. However, there will be seepage forces occurring down to the river bed and pumping action due to turbulent flow around the block protection will induce particle migration. Protection over the full length of the revetment is therefore required if stability of the slope is to be assured. In these circumstances there would appear to be no alternative to the provision of a suitably designed geotextile filter.

Soil Permeability

The permeability test results at 15 m and 30 m depth in six of the boreholes gave coefficients of permeability consistent with SM/SP and SP soils and confirm that they could be dewatered by pumping from deep wells. Down to 10 m, the soils are quite frequently silt and/or clay and dewatering would best be achieved by dewatering the more permeable sands beneath in a sufficiently large area to cause vertical drainage of the silt.

3.5.2 Materials Availability

The only potential construction materials that are readily available in the immediate vicinity are the fine uniformly graded river sand and locally fired bricks. Cement is manufactured in Bangladesh but for the quantities involved and the tight time schedules to be met it will be necessary to depend on imported cement.

River-worn shingle and large cobbles are available from the Panchargarh area in Bangladesh in quite large quantities. Various grades of stone and crushed rock are imported in large quantities from India across the border at Hili. The latter are used by the Bangladesh Railways for track ballast.

The locations of these sources are shown on Figure 3.1.

3.5.3 Morphological

The morphological considerations fall into two main categories:

(a) the range of bed bathymetry that may occur in the immediate vicinity of any training or stabilization works, particularly the maximum scour depth that may develop;

(b) the dynamics of channel and bankline planform, which will determine the location of works, their planform dimensions and the geometry of the structures' terminations.

The first of these categories has been investigated through a combination of bathymetric surveys of areas of interest, notably the priority locations, at different times of the year, 2-D mathematical modelling, physical modelling and analysis of morphological statistics. The scope for statistical analysis is limited by the availability and quality of data but sufficient data has been obtained for setting up and verifying the mathematical and physical models, which can then be used to extend the information to cover specific situations of interest. The 2-D modelling has been used to gain clearer understanding of the processes associated with bend and confluence scour and the bifurcation of flow around chars. Physical modelling is used both to complement the 2-D modelling with regard to the prediction of flow fields and amplification factors and to investigate 3-D dominated effects such as the scour associated with groynes and the stability of structural units.

For the second category, the principal source of information is the analysis of historic records, including maps dating back to 1830, and the more recent satellite imagery. The latter provides an almost sequential data set for the period since 1973, which has proved valuable for char and bend evolution analysis. The maps provide the bankline movement data over the longer period required in order to establish trends that are relevant to the timescale of the priority intervention measures. Further details can be found in Annex 2 of the BRTS Final Report.

3.5.4 Sociological

The sociological impact arising from construction of the Priority Works is described in Chapter 4 and further details may be found in the Environmental Impact Assessment.

Considerations that directly influence the design of the works are:

- alignment of cross-bars to avoid as far as possible resettlement of farmers and fragmentation of land holdings.
- Minimum interference with established land and water access routes, during and after construction.
- Provision of steps built into the revetment to facilitate access to country boats and for general washing and recreational purposes.
- Provision of a refuge berm on the downstream side of the cross-bar where displaced families can establish temporary accommodation.
- Use of productive vegetation for protection against wave and rain erosion to be encouraged.
- Works to be designed for maximum opportunity for employment of local labour during both initial construction and subsequent maintenance.
- Wherever possible hard-points will be located so as to provide direct protection to existing population centres (subject to satisfaction of the primary objective).

3.5.5 Mooring Facilities

Existing arrangements for mooring vessels of all sizes from cargo vessels and ferries to country boats are rudimentary. Ferries have pontoons moored to the bank to provide an intermediate platform of fixed height relative to water level but other vessels typically moor up against the natural bank, wherever the deep water channel happens to be located at the time. At Sirajganj this informal mooring extends from the most downstream portion of the existing bank revetment for a distance of about 1 km.

The standard hard-point design includes country boat steps, and at the continuous revetment at Sirajganj, country boat steps are provided at approximately 500 m intervals in addition to the passenger ferry terminal which will replace the existing facility.

3.5.6 Construction Windows

The design of stabilization and training works must take into consideration their constructability in the Brahmaputra river environment. Since the works will most often be constructed in flowing water in the vicinity of locations where active erosion is taking place, this means that the flow velocities will generally be in the upper quartile. To maintain a high level of construction control in flow velocities which may exceed 3.5 m/s would make the task an order of magnitude more difficult and therefore the construction that much more costly.

The cross-sectional data that has been collected for the BRTS was therefore analysed to determine the period of time during which different flows would not be exceeded for varying probabilities. An example of the output from this analysis is shown in Figure 3.2. From which it can be seen that for a mean velocity of 0.7 m/s, corresponding to a maximum probable velocity of about 1.4 m/s, there is a fairly clear cutoff between 150 and 200 days. In fact it is found that even if a somewhat larger velocity was considered acceptable this would make comparatively little difference to the length of the window.

3.6 Revetment Design Considerations

The main elements of the revetment design are:

- a formed slope that is designed to be stable under normal combined earthquake loading and drawdown conditions.
- A geotextile fabric laid on the slope that is designed to permit drainage of the underlying soil while preventing migration of soil particles under differential pressures induced by wave action, turbulent river currents and soil-water flow.
- An armour layer designed to hold the geotextile firmly in position and therefore capable of resisting the forces induced by high velocity flow and wave action; it must be sufficiently robust and durable to withstanding abrasion due to sediment laden water and inter-block movement.
- An apron, commonly called a falling apron or launching apron, consisting of armour material placed at the toe of the slope; this acts as a stockpile that is drawn upon through a natural bed armouring process when unusually deep scour develops off the toe of the revetment.

The basic principles on which the design of the revetment for the Phase 1 Priority Works are based are set out in the Technical Note issued in January 1992 and reproduced in Appendix C.

An extensive programme of physical model tests was carried out at the RRI facilities at Faridpur to provide information on scour depths and near bank velocities, two of the most crucial design parameters, and to investigate the performance of the apron under a range of conditions. One important conclusion was that the geotextile filter membrane should not be extended under the falling apron.

A further Technical Note entitled "Design Note on Revetments for Priority Works" was issued in May 1992 which brought together the conclusions that arose from the physical modelling programme, the results of some additional geotechnical analysis and the steadily improving level of knowledge regarding the behaviour of the river. This is reproduced in Appendix D.

The background to the derivation of the design criteria is presented in Section 3.8.

3.7 Filter Layer Design

The filter layer laid on the formed bank slope is seen as a crucial element of the revetment and its absence in an effective form is thought to have been a major contributory cause to the failure of many river training works constructed on the Brahmaputra in the past.

3\

Consideration was first given to the possibility of using locally available material to form a stable mineral filter but it was soon apparent that this was impracticable. In order to satisfy the normal filter design criteria, it would be necessary to provide a three zone graded mineral filter. Apart from the material grading quality control difficulties, the placement of a multilayer filter in flowing highly turbid water would present enormous practical problems.

Geotextiles have been extensively used in other countries for this purpose and under similar environmental conditions and good guidelines for the design and construction of revetments incorporating geotextiles have been published by PIANC (1987). Further advice, particularly regarding material properties was sought from leading manufacturers and the conclusions were set out in a BRTS Technical Note on Geotextile Selection issued in February 1992. This is reproduced as Appendix E.

3.8 Design Criteria

3.8.1 Hydrological

The standard hydrological design event is one with a 100 year return period. The definition of such an event has been derived by the Flood Modelling and Management Project (FAP-25) based on the data derived from a 25 year simulation using the MIKE 11 General Model. A closely similar approach was followed using the BRTS Jamuna model, which is a refinement of the General Model, to derive the design water levels at the priority locations with specified confidence limits shown in Table 3.1. This work is described in detail in Part 7 of the BRTS Report on Model Studies.

For the purposes of designing the Phase 1 works, it has been assumed that the Jamuna Bridge will be built and water levels modified accordingly. No direct provision has been made for the possible construction of a left bank flood embankment because of the uncertainty as to its final layout and therefore its influence on water levels; however this possibility has not been ignored and the works have been designed to facilitate modification to accommodate such an increase in water levels when the need arises.

Low Water Level is defined as the lowest annual river level with a 50 percent probability of occurrence, corresponding to a simulated discharge of 2 year return period. For practical purposes the water level of greater relevance is that which will not be exceeded for a specific degree of probability over a reasonable construction period. For the design of the works this level has been taken as LWL+2m, which corresponds approximately to a 50 percent probability of exceedance within a 160 day window. In practice this means that there is a 50 percent probability that work can be carried out in the dry over a continuous period of 160 days.

3.8.2 Hydrodynamic

One of the most important design parameters for the bank stabilization works is the maximum near bank velocity. This typically is the ruling criterion for the sizing of the armour layer, the other criterion being resistance to wind induced wave action.

Based on the results of a number of physical model tests and interpretation of the 1-D and 2-D mathematical model results it has shown that there is a relationship between the mean

Table 3.1 Preliminary Estimate of Confidence Limits for Predicting Water Levels at Priority Sites

Potential Sources Of Error	Comment	Possible Confidence Limits Fulchari	Sariakandi	Mathurapara	Kazipur	Sirajganj	Betil	Likely Confidence Limits Fulchari	Sriakandi	Mathurapara	Kazipur	Sirajganj	Betil
Cross Section Data													
1.1 Incorrect datum	Dependent on distance to SOB BM	.02	.08	.08	.06	.06	.08	.01	.03	.03	.03	.03	.03
1.2 Location error	Only minimal local effect	.01	.01	.01	.01	.01	.01	.00	.00	.00	.00	.00	.00
1.3 Survey error	Only minimal local effect	.01	.01	.01	.01	.01	.01	.00	.00	.00	.00	.00	.00
1.4 Selection of cross section	Minimal; overall effect averaged	.01	.01	.01	.01	.01	.01	.00	.00	.00	.00	.00	.00
1.5 Combination		.02	.08	.08	.06	.06	.08	.01	.03	.03	.03	.03	.03
Water Level Gauge													
2.1 Missing data	Identifiable	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
2.2 Static water level	Identifiable	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
2.3 Reading error	Often Identifiable	.10	.10	.10	.10	.10	.10	.05	.05	.05	.05	.05	.05
2.4 Gauge level incorrect	Poor transfer of level from BWDB BM to gauge	.10	.10	.10	.10	.10	.10	.05	.05	.05	.05	.05	.05
2.5 Gauge sited incorrectly	Not located in recorded position	.10	.10	.10	.10	.10	.10	.02	.02	.02	.02	.02	.02
2.5 Cross flow induces a WL slope and affects WL reading	Cross flows not evident at these sites	.10	.10	.10	.10	.10	.10	.00	.00	.00	.00	.00	.00
2.7 Transfer of level to BWDB BM	Dependent on traverse distance (consider av of adj gauges)	.02	.03	.03	.03	.03	.03	.05	.08	.08	.06	.06	.08
2.8 Combination		.10	.10	.10	.10	.10	.10	.05	.08	.08	.06	.06	.08
Assumptions In 1-D Model													
3.1 Constant Mannings "n" used	Variation with depth effectively covers seasonal variation	.10	.10	.10	.10	.10	.10	.03	.03	.03	.03	.03	.03
3.2 Fixed cross sections	25 year runs indicates this is OK	.20	.20	.20	.20	.20	.20	.05	.05	.05	.05	.05	.05
3.3 Flow into model in error	Variation in inflow	.50	.45	.45	.45	.38	.32	.17	.15	.15	.15	.13	.11
3.4 Bahadurabad gauge datum inconsistent with RB sites	FAP-18 due to link left and right banks	.80	.00	.00	.00	.00	.00	.50	.00	.00	.00	.00	.00
3.5 Combination		.80	.45	.45	.45	.38	.32	.50	.15	.15	.15	.13	.11
Topographic Surveys													
4.1 SOB bench marks	Possible error in SOB bench marks	.05	.05	.05	.05	.05	.05	.02	.02	.02	.02	.02	.02
4.2 Transfer of levels	Dependent upon distance from SOB BM	.02	.08	.08	.06	.06	.08	.01	.03	.03	.03	.03	.03
4.3 Combination		.05	.08	.08	.06	.06	.08	.02	.03	.03	.03	.03	.03
Assessment Of Return Period													
5.1 Representativeness of 25 year	Sampling properties of 25 year period	.30	.30	.30	.30	.30	.30	.20	.20	.20	.20	.20	.20

Notes : BM - Bench Mark
WL - Water Level
RB - Right Bank

velocity in the river section as a whole and the maximum velocity that can develop near the bank and around artificial projections such as groynes. The derivation of these values is described in Appendix F and the results are set out in Table 3.2.

Table 3.2 Design Velocities

(Probability for exceedance of 100 year design event is 1% in project lifetime (30 year)).

Type of Structure	Amplification Factor	Design Velocity (m/s)
Revetment straight section	1.1	3.7
Revetment upstream termination	1.3	4.4
Head of groyne*	1.4	4.8

* including upstream termination at Ranigram Groyne, Sirajganj

The revetment material size of the protective layer must satisfy the following relationship, referred to as the JMBA equation:

$$D = \frac{0.7\,v^2}{2(S_s-1)g} \cdot \frac{2}{\left\{\log\left(\frac{6\,h}{D}\right)\right\}^2} \cdot \frac{1}{\left\{1-\frac{\sin^2\phi}{\sin^2\Theta}\right\}^{1/2}}$$

Where:

D = dimension of cube (m)
v = maximum velocity close to bank (m/s)
S_s = Specific gravity of cube material
= 1.98 for concrete with brick aggregate
g = gravitational acceleration
= 9.81 m/s^2
h = depth of water (m)
ϕ = bank slope (degree)
Θ = angle of internal friction of slope (degree)
= 40°
$\frac{h}{D}$ = 5, shallow water condition, is assumed.

34

To simplify construction control, the armour layer has been divided into three categories. From considerations of flow velocity and wave action (see the following section) the majority of the length of bank parallel revetment will be provided with armour consisting of 550 mm brick aggregate concrete cubes (or the equivalent in stone aggregate concrete blocks or rock as appropriate). The part of the upstream revetment termination that is exposed to the highest velocities, and the first 100 m of straight revetment immediately downstream, will be provided principally with 720 mm block armouring and the noses of groynes will be provided with 850 mm block armouring.

Concrete block sizes using the stone aggregate alternative are calculated in exactly the same way but using the specific gravity for concrete with stone aggregate (2.3) resulting In the corresponding block sizes of 450 mm, 550 mm and 650 mm.

Rock sizes are calculated by a similar methodology, resulting in D_{50} equivalent to 320 mm, 450 mm and 530 mm respectively. The revetment profile using the rock alternative retains concrete blocks on the upper part of the revetment slope (above LWL + 2 m) and hence rock sizing is not affected by wave action. The calculation of rock sizes is given in Appendix G.

During physical model testing of hard-point structures, the highest velocity consistently occurred in an area starting from the transition from the conical termination to the straight section and about 50 m downstream thereof. Similarly, during physical model investigation of local scour around structures, it was found that the maximum scour could occur at the upstream termination and/or about 100 m downstream. Hence the heavier section of the upstream termination is retained for the first 100 m of linear revetment (see Figure 3.3).

3.8.3 Wind wave action

The second most important criterion governing the size of the material in the protective layer for stabilization and training works is that the material must be able to resist the forces induced by wind wave action. The formula being used is that developed by Pilarczyk.

$$D = \frac{H_s}{S_s-1} \cdot \frac{1}{\beta} \cdot \frac{\sqrt{E}}{\cos\phi}$$

Where:
D = cube dimension (m)
H_s = significant wave height (m) = 1.0 m
S_s = specific gravity of revetment material
= 1.98
β = strength coefficient
= 3 for cubes
= 2 for randomly dumped cubes

E = wave breaking parameter
$= 1.25 \dfrac{T \tan\phi}{\sqrt{H_s}}$

T = Wave period (s) = 3.0s
ϕ = bank slope (degrees) = 16°

A considerable amount of data collection and analysis was carried out as a part of the JMB design studies; this has been reviewed and some further analysis carried out on data from five meteorological stations, including Sirajganj, obtained from the Bangladesh Meteorological Department (see Appendix H).

The screening of the data disclosed a number of errors which were corrected by the Meteorological Department. The possibility of further less obvious errors cannot be excluded. A reasonable consistency was found between the five sets of data and the following conclusions were reached:

- wind speeds exceeding 22 knots occur only once every 2 to 4 years;
- high winds are most frequent in the pre- and post-monsoon period but can occur in January, February and June to September;
- maximum speeds can occur with winds from almost all directions;
- a speed of 35 knots (18.0 m/s) has a return period of about 5 years.
- The JMB statistics are considered to probably overestimate wind speeds by about 25 percent but are otherwise applicable to the Brahmaputra as a whole.
- Squalls tend to have a very short duration, the peak lasting from a few minutes up to 20 minutes for the records examined; a design duration of 30 minutes has therefore been adopted as being reasonably conservative.
- Only winds blowing from the north-east to south-east quadrant can produce waves affecting works on the right bank of the Jamuna; this reduces the frequency of occurrence of significant winds.
- Estimated effective fetches lie in the range of 5,000 m at high flow to only 2,300 m at low flow.
- The design wave height has been found to have a height of 1.0 m and a peak period of 3.0 s.
- This assessment does not take current into consideration; velocity and turbulence is believed to have a damping effect on short waves but it is not possible with the present state of knowledge to quantify this.

3.8.4 Geotechnical

The relatively uniform soils found along the full length of the right bank within the study area have been described in outline above. The vertical stratification for design of the Priority Works has been assumed to be as shown in Table 3.3 below:

36

Table 3.3 Soil Stratification

Depth (m)	US Soil Classification	Description
0-10	CL,ML	Clays & silts of low plasticity, non-plastic silts
10-20	SM	Silty fine sands (non-plastic)
20-30	SP-SM,SP	Slightly silty medium fine (predominantly fine) sands, medium-fine (predominantly fine) sands (all non-plastic).

As described in Appendix I, the stability of the revetment section has been checked by slip circle analysis for both static and seismic conditions. In accordance with recommendations by Seed that for the purposes of pseudo-static analysis, horizontal accelerations of 0.10 g and 0.15 g, used with a safety factor of 1.15, could be taken as representative of earthquakes of magnitude 6.5 and 8.2 respectively, an acceleration of 0.15 g was used for this analysis. As, however, the design earthquake is magnitude 7, a reduced factor of safety of 1.1 is acceptable, or 1.05 for a combination of extreme events. For static loading conditions, including the scoured face at the foot of the revetment, a factor of safety of 1.5 was applied.

Shear strength criteria have been selected on the basis of the site investigations. They are lower than the shear strength assumed for the Jamuna Bridge design on the basis of triaxial tests, but it is considered that this is justified by the universal nature of the revetment design and the need to have a stable revetment where ground condition might not have been specifically investigated.

The apron setting level has been determined from a combination of slope stability considerations, bearing in mind that the scour slope will be steeper than the profiled revetment section, and a reasonable maximum dredging depth. From the results of numerous physical model tests and reported experience in India and elsewhere, it has been concluded that it is reasonable to assume that the slope of the launched apron is unlikely to be steeper than 1V:2H. With this configuration and the base of the apron set at 10 m below LWL it was found that a lower revetment slope of 1V:3.5H gave acceptable factors of safety. The upper slope, above LWL+2m, had to be flattened to the same slope of 1V:3.5V from the earlier design of 1V:3.0H following analysis of the piezometer data collected at Sirajganj over the full annual cycle of water levels. These data showed that during the falling stage the piezometric head in the more clayey/silty strata of the bank could be as much as 1 m higher than the river water level for considerable periods of time. Further details of this and other geotechnical criteria are to be found in Appendix I.

Five sections have been analysed (see Figure 3.4) of which two, BRT 1 and BRT 2 for low and high water level respectively, most closely resemble the design sections for the Priority Works. MAT01 and MATR1 assume no dredging is carried out and the apron is dumped on an existing high river bed level. BRT 3 includes the effect of a 3.5 m high embankment on the section.

Three series of runs were carried for these sections. The first, BP_{old}, undertaken on a main frame computer in Halcrow's head office at Burderop Park in March 1992, has been superseded by the two subsequent runs, BP_{new} (Burderop Park) and BRTS (Dhaka).

The results are shown in Table 3.4. BP_{old} is included in Table 1 only because it is those results which are given in Appendix I referred to above. With one (non-critical) exception, BRTS and BP_{new} show very good correlation.

Table 3.4 BRTS SLIP5 Analysis

Sections	Sections of slope	Factor of Safety Static Loading			Factor of Safety Seismic Loading 0.15g		
		BRTS	BP_{new}	BP_{old}	BRTS	BP_{new}	BP_{old}
MATO1	Revetment Scour Overall	1.69 1.43 1.57	1.70 1.44 1.56	1.79 1.48 1.67	1.06 1.04 1.07	1.04 1.03 1.07	1.15 1.02 1.17
MATR1	Revetment Scour Overall	2.12 1.43 1.68	- - -	2.27 1.48 1.87	1.29 1.04 1.14	- 1.03 -	1.36 1.02 1.30
BRT1	Revetment Scour Overall	1.79 1.52 1.80	1.79 1.51 1.80	1.85 1.61 1.91	1.17 1.10 1.18	1.19 1.10 1.19	1.18 1.10 1.31
BRT2	Revetment Scour Overall	2.16 1.52 1.93	2.21* 1.51 1.92	2.36 1.61 2.17	1.36 1.10 1.27	1.40 1.10 1.27	1.40 1.10 1.45
BRT3	Revetment Scour Overall	1.60 1.52 1.76	- - -	1.60 1.61 1.86	1.06 1.10 1.16	1.06 - 1.16	1.05 1.10 1.27

* data input error

3.8.5 Morphological

The morphological characteristics of the river that are of most direct concern for the structural design bank stabilization works are:

(a) the depth of scour that may be expected to occur in the immediate proximity of the structure, during its design life, with an approximately 1 percent combined exceedance probability. This will determine the apron geometry and influence its setting depth.

38

(b) The planforms of embayments that may develop on the upstream and downstream sides of a hard-point or groyne and the velocity field associated with these. These will determine the planform geometry of a hard-point and its associated cross-bar.

Scour Depth

The assessment of maximum scour that can occur has been the subject of several complementary studies:

- physical model studies, using mobile bed conditions to investigate scour associated with groynes and other active forms of training works;
- river surveys of locations were active bank erosion is taking place;
- 2-D mathematical model simulation;
- morphological model simulation for simple bends;
- analysis of BWDB river cross-sectional data surveyed over a period of about 30 years;

Analysis of these studies points to the conclusion that the worst scour that can occur in the study reach arises from exceptional combinations of conditions associated with major anabranch confluences. The next most severe condition is associated with vortex shedding downstream of a groyne nose.

It has been concluded that satisfactory values for design purposes are 29 m for scour at the toe of a bank parallel revetment, 33 m at the nose of a groyne and 33 m at the upstream termination of a bank revetment, measured below the 100 year design flood level. The falling apron is designed to distribute the equivalent of at least two layers of armour material over the deformed slope face with full scour development. Model tests and experience indicate that the redistributed armour material forms a remarkably uniform single layer and it is reasonable to expect that there will in practice be an in-built reserve of material that can be drawn upon in the unusual case of scour exceeding these design values. The apron design adopted will in theory provide sufficient armour material for a single layer over the complete surface for scour depth of 44 m at the nose of a groyne and 33 m at the toe of straight revetment.

The second of the morphological related criteria for design presents the harder problem because of the ever changing conditions on the river and the absence of clear behavioural trends. The location and planform of the works must take into consideration not only the present conditions but what may be expected to occur within the planning horizon for the works concerned, which will normally be 20 to 30 years.

Embayment Planform

In general the location of a hard-point and the length of its revetment will be determined by two considerations:

39

- providing direct protection to a specific object, such as a substantial town or major infrastructure;

- minimizing the risk of outflanking.

Both these are closely related to the form and depth of embayment that could form. The location and length of the hard-point must be such that protection is provided even with full embayment development, or at least such that relatively low cost secondary works (for example the use of porcupines) would be sufficient to prevent development of the embayment beyond certain limits. Equally important is that the cross-bar should remain functional under similar conditions.

There is very little data on which to base an estimate of the probable maximum embayment development upstream of a hard-point. The only directly relevant example is the existing revetment at Sirajganj, where the embayment has not, as yet, evolved into the re-entry planform that is a feature of meandering rivers that encounter natural hard-points, such as rock outcrops. It cannot however be safely assumed on the basis of this very limited data that such forms will not evolve on the Brahmaputra.

The approach adopted has been to take the worst anabranch bend/embayments picked from the available satellite imagery and to use these as examples of the aspect ratio (depth to chord length) of a simple, approximately symmetrical, embayment that can be expected to develop anywhere on the right bank. The bends at Jalalpur and Fulcharighat that developed between 1984 and 1990 appear to be unusually severe cases and are appropriate for this purpose.

The shape of the re-entry that could occur upstream of a hard-point is largely a matter of conjecture. There is one known example of a large anabranch thalweg on the left bank that in 1992 formed an exceptionally tight bend with a greater than 180 degree change of flow direction. This shows that such features can physically exist in the Brahmaputra environment but they are rare. It is more likely that a severe re-entry will be associated with a smaller channel, which is more likely to develop the shorter wavelength and high sinuosity bends that are needed for such a feature to evolve. Examples of such channel forms have again been picked from the satellite imagery and a composite embayment planform derived.

In the BRTS hard-point design, the junction between revetment and cross-bar is at the downstream termination; the cross-bar is angled in the downstream direction towards the flood embankment (see Figure 3.5). For a given re-entry, therefore, the length of revetment necessary to provide adequate protection to the cross-bar against erosion is kept to the minimum.

The composite embayment planform which has emerged from this exercise is consistent with the indications obtained from the physical model tests carried out as a part of the JMB Feasibility Study and the recorded observations of such features on meandering rivers. Although it is generally accepted that physical models, because of scaling effects, are not a reliable means of quantifying bank scour, they will provide a picture of the velocity fields that can develop and therefore some indication of the likely worst planform evolution. Taking all these indications together the design criterion adopted would seem to be, if anything,

48

conservative. At worst, relatively low cost measures should suffice to prevent further development of the re-entry.

3.9 Revetment Structures

3.9.1 Layout in Plan

For the reasons described above hard-point revetment structures are the principal form of river bank stabilization measure proposed for the short term and long term works.

The layout of a typical hard-point is shown in Figure 3.5. The length of straight revetment is such as to ensure that there will be an acceptably low risk of the embayment, or any re-entry upstream of the hard-point, threatening the integrity of the cross-bar. The cross-bar is angled back in plan to decrease further the risk of erosion from the upstream side. The cross-bar is lightly armoured against wave action from out-of-bank flow by brick mattressing on its upstream side, for the whole length back to the BRE, and for the 100 m nearest to the river bank on the downstream side.

The revetment is returned by upstream and downstream terminations. As explained earlier, the upstream termination and the length of straight revetment immediately downstream of it will be subjected to higher flow velocities than the rest of the straight revetment and the downstream termination, and hence heavier armouring is provided as shown in Figure 3.3

The layout at Sirajganj (Figure 2.2) with more than 2 km of standard revetment, is a special case. It has a conventional downstream termination, but the upstream termination, which incorporates the existing Ranigram Groyne, will be armoured as appropriate to the nose of a groyne.

3.9.2 Typical Revetment Section

The essential features of the revetment to be determined by design are:

Crest level
Slope
Geotextile filter membrance
Armouring
Apron

The basic principles on which the design of the revetment for the Phase 1 works are based are set out in Appendices C and D. A typical revetment section in shown in Figure 3.6.

The crest level is determined from the 100 year return period water level, derived from the 1-D hydrodynamic modelling programme described in Part 7 of the Report on Model Studies, with a 1.0 m allowance for freeboard. A single layer of placed armouring extends from the concrete crest wall to the lowest water level likely to be experienced during construction, LWL + 2 m. From that level to the apron setting depth, armouring will be dumped to form the equivalent of two layers.

91

The apron setting level has been determined from considerations of slope stability and dredging depth, as explained in Section 3.8.4. The quantity of armour material in the apron is adequate to protect the scoured slope to design scour depth with a thickness of armouring in excess of two layers.

There is strong evidence that a geotextile membrane under the apron would interfere with the launching mechanism leaving the geotextile exposed. It is therefore not continued under the apron beyond the first 2 m.

Based on the flow velocity and wave considerations described above, the following sizes have been determined for concrete cubic block armouring using brick aggregate:

Noses of groynes:	850 mm
Upstream terminations:	720 mm
Linear revetments and downstream terminations:	550 mm

Cross sections through the crest and cross-bar are shown in Figure 3.7.

3.9.3 Selection of Armour Material

The revetment design provides for a choice to be made between three different armour materials, namely:

- concrete blocks with brick aggregate
- concrete blocks with stone aggregate
- quarried rock

Concrete cubes with brick aggregate constitute the traditional armour material used in river training works hitherto in Bangladesh. Of the constituents, the formwork, sand and coarse aggregate are all available from sources within Bangladesh and Brahmaputra water is suitable for concrete. The continued use of brick aggregate in preference to crushed stone would be expected to result in lower overall cost, but the impact of the strict environmental management controls that will be imposed on contractors with regard to the use of brick, and the effect of the relatively high demand on the market price, may alter this situation. Some coarse aggregate stone is imported from India and some is available from within the country at Sylhet. In view of the critical importance of timeous supply of materials during revetment construction, it assumed that the cement will have to be imported.

The quantities of rock which would be required, however, indicate that to meet demand, it would have to be imported. Similarly, indications are that local supplies of river boulders would be inadequate to meet the demand. Dependency on imported rock would, however, place a high premium on the continuation of an open border, and could provide suppliers with an undesirably strong base for price negotiation.

The Phase 1 works are designed with concrete block armouring (brick or stone aggregate) above LWL + 2m, and alternatives of concrete blocks or rock below that level. Hence three

42

alternative tenders are required (see Annex 6) - concrete blocks with brick aggregate, concrete blocks with stone aggregate, and quarried rock (below LWL + 2 m with the cheaper type of concrete block above).

This arrangement provides the tenderers with the flexibility to offer any particular comparative advantage that they may have with respect to their access to specialized plant or markets. Also although one of the three materials will be selected at the time of tender evaluation, the other two alternatives will offer a useful basis for a variation should circumstances change and the first selection prove untenable.

The same procedures may be adopted for the long term works, unless a clear trend towards a particular alternative emerges in the meantime.

3.9.4 Principal Quantities

The principal quantities required for construction of the revetment at Sirajganj, reconstruction of Kalitola Groyne, and a typical hard-point, using brick aggregate concrete block armouring, are given in Table 3.3.

Table 3.5 Principal Quantities of Phase 1 Works

Description	Unit	Quantity		
Location		Sirajganj	Kalitola Groyne	Typical Hard-point
Dredging	m^3	2,200,000	30,000	300,000[(1)]
Hydraulic Fill	m^3	1,650,000	65,000[(2)]	50,000[(1)]
Geotextile	m^2	190,000	31,000	66,000
550 mm armour blocks	no	900,000[(3)]	70,000[(3)]	177,000
720 mm armour blocks	no	0	0	83,000
850 mm armour blocks	no	48,000	43,000	0
Brick mattress on cross-bar	m^2	0	3,000	17,000[(4)]
Cross-bar embankment	m^3	0	0	110,000[(4)]
Asphalt road	m^2	11,000	0	0
Brick surfaced road	m^2	0	1,000	2,500[(4)]

Notes
(1) These quantities will vary considerably from site to site
(2) Including trucked - in fill
(3) Including blocks which may be recovered
(4) Depending on length of cross-bar

4B

4. DESCRIPTION OF THE PHASE 1 WORKS

4.1 General

As already described in section 2.1.4, the Phase 1 works include three stages of implementation, the first, Phase 1A, being the Priority works to be constructed at Sariakandi, Mathurapara and Sirajganj under the River Bank Protection to be financed by IDA. Phase 1B will comprise measures to stabilize the reaches immediately north of the Phase 1A locations, at two locations north of Sariakandi, and two locations north of Sirajganj. Phase 1C will comprise bank protection at the three remaining priority locations, namely Fulcharighat, Kazipur and Betil.

Apart from Sirajganj, the river bank protection measures to be constructed under Phase 1 of the Master Plan consist of hard-point structures with 300 m long straight revetments. Draft tender documents for construction at Sariakandi and Mathurapara (Contract No. B1) and Sirajganj (Contract No. B2) were issued in August 1992. The tender documents for the remainder of Phase 1 (Annex 6) are based on two hard-points per contract, one at each of the two locations north of Sariakandi and the two locations north of Sirajganj, and two at each of the three Phase 1C locations.

4.2 Sirajganj

4.2.1 The Design Concept

At Sirajganj the erosion of the bankline since the 1950s has resulted in parts of the old established town now fronting directly onto the river bank. In other areas urban and peri-urban development has expanded up to the river bank. The existing bank protection works, consisting of concrete block armouring but without an effective underlying filter layer, extend over a length of about 1.5 km and when this was threatened with outflanking in the 1980s, Ranigram Groyne was constructed to the north of the town. The groyne has a sand core with concrete block armour but no filter layer. The total length provided with direct protection by these existing works is some 2.2 km.

It is very apparent from the 1992 bankline planform that this stabilized length of bankline forms a modest protrusion into the river. For reasons that are not clear, this is also the point on the river where the braiding intensity reduces markedly and linked to this is the fact that the main river channel tends to be more pronounced and somewhat less shifting than further upstream. In recent years the effect has been further enhanced by the gradual decline of the main left bank anabranch which has resulted in a larger proportion of the river flow passing down the right bank anabranch past Sirajganj.

The existing Sirajganj bank protection works are thus becoming increasingly exposed to steadily more severe attack by the river and since even normal monsoon season flow conditions result in local failures, due both to undermining and the large scale migration of the fine sandy material through the coarse matrix of armour layer. A larger than normal flow is likely to cause widespread failure that could not be contained with the existing facilities available to the concerned authorities. Rapid erosion of the bankline would follow and much of the town could be destroyed in a matter of 5 years.

44

The immediate object in this case is to stabilize the existing bankline in the vicinity of the Sirajganj urban area to a standard that will withstand at least a 1 in 100 year event. Larger events would possibly cause significant damage requiring urgent attention but would not be expected to result in catastrophic failure.

There are strong social arguments for providing the upgraded standard of protection to the full length of bankline that is currently protected, although in purely economic terms the most downstream length perhaps ranks lower in importance.

Considerations affecting the selection of the form of protection upgrading included:

(a) lack of confidence in the capability of Ranigram Groyne to remain structurally stable if exposed to a large flood flow (both shank and nose have significant design deficiencies);

(b) the practical difficulties relating to the assessment of the physical condition of the underwater portion of the existing bank revetment, much of which is buried by shifting sandbars;

(c) the more predictable and cost effective performance of bank revetment over multiple groynes as a means of stabilizing the bankline under the hydraulic and morphological conditions experienced at Sirajganj.

The conclusion was that the most appropriate approach would be to upgrade the existing revetment, by:

- salvaging all reusable material;
- building up the bank profile to a geotechnical stable slope;
- placing a modern purpose designed geotextile filter;
- placing an armour layer on the slope;
- forming an underwater toe apron that would provide a source of armour material for distribution by natural forces in the event of occurrence of severe scour.

To overcome the uncertainty over the reliability of the Ranigram Groyne, the revetment will be extended upstream to connect with the groyne and a new nose will be constructed.

4.2.2 Layout of the Works

The works at Sirajganj will consist of the following main components, as can be seen from Figure 2.2.

95

Section A - Upstream Termination incorporating upgrading of the nose of Ranigram Groyne. The present condition of the nose is poor with oversteep slopes. The absence of a proper filter under the armour means that failure can occur very rapidly once instability is induced.

Section B - Land reclamation behind (C) over an area of about 25 ha. The main object of this landfill is to provide a backing to Ranigram Groyne to safeguard against possible piping failure.

Section C - Length of new revetment spanning between Ranigram Groyne head and the upstream limit of the existing town protection revetment, a distance of about 700 m. It is more cost-effective to provide this length of revetment than to provide downstream strengthening of Ranigram and a full upstream termination for town revetment. The provision of security to the reclaimed area is an added benefit.

Section D - Replacement of the existing revetment over a distance of about 1,500 m, and a downstream termination. The existing revetment can be considered as four reaches with distinctive features:

(1) the upstream 250 m which comes under active attack even at moderately low flows;

(2) the next 500 m where the bank slope is relatively gentle above LWL but unstably steep below the low water line;

(3) the next 500 m where the upper bank slope is relatively steep and in planform there are a series of small embayments and headlands; this section includes the severe embayment at the old Jail;

(4) the downstream 250 m which in planform curves gently landward and which has a generally more stable appearance; over the last 100 m the revetment is partially buried by accreted land.

The overall bank slope along the full length of the existing revetment is potentially unstable under even modest earthquake loading. Any additional scour is likely to result in local collapse of the type that resulted in the Jailkhanna embayment, which could rapidly develop into a major progressive erosive failure. None of the existing revetment has an effective filter layer under the armour and as such is vulnerable to soil migration failure induced by high velocity river flow or seepage flow in the bank during the falling stage.

4.3 Sariakandi and Mathurapara

4.3.1 The Design Concept

In this case the primary objective is distinctly different to that at Sirajganj; although the associated protection of the small town of Sariakandi has certain similarities.

46

Over the past 30 years the right bank of the Jamuna (Brahmaputra) has shifted westward by a net amount in the order 3 km over a considerable length and there is now only a thin sliver of land separating it from the Bangali River; in places this strip is less than 1 km wide. Erosion is continuing and during the last monsoon season the bankline retreated a further 300 m on average over a length of several kilometres, further reducing the separation. Since aggressive bends can result in annual rates of bank erosion in the range 500 to 1000 m, and occasionally more, it is reasonable to anticipate an imminent breakthrough, although it is not possible to predict the particular year in which it will occur.

The meandering pattern of the Bangali means that the width of the separating band of land varies and therefore the most vulnerable stretches, in terms of the least amount of erosion required for a breakthrough, can be simply identified. However the pattern of the Jamuna bank erosion is of a complex stochastic nature and it is not necessarily most likely that the breakthrough will first occur at the narrowest point. The least width of separation at present is in fact to the north of Sariakandi on a stretch of the Jamuna river that has been relatively quiescent for a number of years and which at present shows no signs of becoming active again. Although by the nature of the river, this situation could change rapidly.

The object in this case is therefore to stabilize the bankline in the most cost-effective way that will minimise the risk of a breakthrough occurring. In order to achieve this it is necessary to look at the reach as a whole and not to be unduly influenced by the variation in the width of the separating strip or the pattern of erosion that happens to be taking place in any one year. At the same time the presence of significant townships such as Sariakandi cannot be ignored.

The fundamental assumption that has been made, based on the interpretation of planform dynamics, is that it is unlikely that severe erosion will occur simultaneously to the north and south of Sariakandi and that although the focus may switch, this will take place over several years so that an appropriate timely response can be initiated.

The object of the Priority Works is therefore narrowed to the stabilization of the reach south of Sariakandi and the protection of the township itself.

The second important assumption is that the depth of embayments that will form between hardened lengths of the river bank will not exceed the most severe depth of penetration observed in the case of very aggressive bends. This assumption is also consistent with the conclusion reached by the JMB detailed design studies which was based on both morphological interpretation and physical modelling.

What remains largely a matter of conjecture is the worst re-entrant configuration that can occur immediately upstream of a hard-point. It is this form of erosion that can potentially result in outflanking and effective failure of the hard-point. The most conservative approach would be to follow the methodology commonly employed in India for the determination of the length of guide bunds for barrages. The application of such a procedure in this case would however result in excessively long hard-points that would be out of proportion to the benefits involved.

The rational argument for modifying the Indian practice in this case is that the probability of occurrence of such severe conditions, although not readily quantifiable, is certainly low. It is therefore more cost effective to accept this low risk and to plan for remedial action to be taken, in the form of some additional stabilization to control the development of the embayment, should the need arise.

The likely worst planform of the embayment has therefore been estimated on the basis of:

- the most severe embayment planform observed in nature, using the satellite imagery dating back to 1973;
- the results of the JMB physical modelling;
- examples of similar embayment developments associated with hard-points, usually naturally occurring, on rivers in other countries;

Given these considerations the spacing of the hard-points was selected in order to prevent the estimated embayment intersecting the Bangali River and the lengths of the hard-points was set to accommodate the assumed worst planform development as described above.

To provide a level of security to the township of Sariakandi equivalent to that provided for Sirajganj would be out of proportion, and economically unjustifiable, but some upgrading of the rather poor protection provided by the existing Kalitola Groyne is appropriate, particularly if it can be incorporated into the plan for primary reach stabilization.

The resulting configuration was therefore based on the upgrading of Kalitola Groyne to form a small hard-point and the provision of two further hard-points which are referred to by the names of Sariakandi and Mathurapara for convenience, although they will most probably be situated some distance from the main population centres associated with these names. Kalitola Groyne is not under severe attack at present but this situation is unlikely to persist for more than a matter of years (a tentative estimate of 5 years has been assumed) after which it is anticipated that further hard-points will be required to the north of Sariakandi (see Figure 2.4).

Because of the ever-changing bankline planform, the final positioning of the hard-points will have to be made close to the time of construction. It is even conceivable that if the focus of erosion shifts to north of the town during the coming two monsoon seasons then the priority may equally shift to the north, necessitating a more radical review of the respective priorities to be assigned to the hard-points.

4.3.2 Layout of the Works

Attention has earlier been drawn to the fact that the location of the hard-points and associated works will have to be reviewed in the light of morphological changes that take place during the monsoon seasons prior to construction. Following changes in location, some modification of the layouts may also be required.

As can be seen from Figure 2.3, the Priority Works at Sariakandi and Mathurapara comprise:

(a) Upgrading of the existing Kalitola Groyne. The present condition of the nose and upstream face of the shank is poor with oversteep slopes and no effective apron to maintain scour at a safe distance. The combination of the absence of a proper filter under the armour and undermining of the face due to the inadequate apron is probably responsible for the two large bowl shaped failures clearly visible on the upstream face of the shank. Other visible evidence of under-design is the displacement of the concrete blocks at the nose, which indicates that they are under-sized.

(b) Construction of two new hard-points 2.5 and 5 km downstream of Kalitola Groyne. Both will consist of a length of standard revetment constructed along the existing bankline which will be wrapped round landward at the upstream end, and to a lesser extent at the downstream end, to form terminations designed to prevent the hard-point from being outflanked in the event of continuing by bank erosion. The straight length of revetment will be 300 m long in each case.

(c) Construction of low earthfill embankments, referred to as cross-bars, linking the hard-points to the main BRE embankment. The purpose of these is to prevent flow passing between the hard-point and the BRE when the river is out-of-bank and the possible consequential erosion of a channel outflanking the hard-point. The cross-bar embankments will be provided with wave and low velocity current protection on the upstream face and the 100 m nearest to the river on the downstream face, consisting of wire covered brick mattressing laid on a geotextile cloth.

4.4 Simla and Sailabari Groyne

4.4.1 The Design Concept

From examination of LANDSAT imagery it can be seen that accretion in the large embayment upstream of Sirajganj started in about 1981 and by 1985 the embayment was almost completely filled in. This timescale for rapid accretion is consistent with the predictors. Since 1986 there has been no significant erosion but there was a short burst of intense erosion immediately downstream in 1988-90 and incipient erosion in the bay downstream of Ranigram Groyne in 1992. The fact that the latter came to nothing is most probably linked to the generally relatively quiescent conditions since 1990 as the river recovers from the 1988 scour and its immediate aftermath. This quiescence has been accentuated by the relatively low monsoon flows over the past two seasons. However, the likely scenario that islands D and E will merge, as descried in section 4.6.1 which follows, may well lead to the bank between Kazipur and Sirajganj coming under increasing attack.

A new hard-point at the site of the existing Sailabari Groyne (Figure 2.5) will have a significant role to play in preventing the formation of any aggressive shorter wavelength bend which would threaten the flank of Ranigram Groyne, and, consequently, Sirajganj town.

To stabilize this reach of the river and prevent any outflanking tendency, a hard-point further north is required. The additional hard-point proposed at Simla will have the further advantage of affording some protection to Simla/Sonali Bazar.

4.4.2 Layout of the Works

Two hard-point revetment structures, each with linear revetment 300 m long and upstream and downstream terminations, will be constructed, one at Simla and one at Sailabari Groyne.

49

These will be similar to the hard-points described in Section 3.9 and will be linked back to the BRE by means of cross-bars. At Sailabari, the existing groyne structure will, as far as possible, be incorporated in the cross-bar of the hard-point.

4.5 Sariakandi North and Naodabaga

4.5.1 The Design Concept

The average rate of bank erosion varies over the length of the river from almost zero in places to more than 100 m/y in the reach starting north of Sariakandi and extending to Sirajganj. However there is firm evidence of rates higher than 600 m/y on the right bank of the river, associated with aggressive anabranch bends, and rates of 200 to 300 m/y can be sustained for several years on end. There are reports of erosion peaking at 1,000 m/y and one such case has been observed on the left bank of the river during the period 1990 to 1992.

Processing of satellite images for the period 1973 to 1992 by FAP-19 has produced data that have been further analysed by BRTS and compared with information derived from older maps.

The records show that a 23 kilometre reach of the right bank in the vicinity of Sariakandi and extending from northing 765000 (y-coordinate of the transverse Mercator Projection of Bangladesh) in the north to 742000 in the south, has suffered relentless erosion since 1956. The overall average rate of erosion for the reach over the last 36 years is 116 m/y and erosion has been fairly evenly spread along the entire 23 km reach during this period.

However, the distribution of erosion over shorter periods shows marked and possibly significant variability. During the period 1956 to 1973 erosion was centred on Sariakandi, but then the focus moved further north, to concentrate erosion along the bank to the east of the village of Naodabaga. As a result, by 1990, the point where the bankline was actually closest to the channel of the Bangali River was north of Sariakandi. In the last two years erosion has ceased in the northern half of the reach; however severe erosion has continued in the southern half of the reach around Mathurapara.

The current erosion rate in the embayment that is forming to the south of Sariakandi is of the order of 200 m/y. As a result the area where the Brahmaputra and the Bangali are closest is now near Mathurapara, where only about 1 km now separates the river channels. Hence, it is at Mathurapara that the risk of a breakthrough is highest at the moment and accordingly this is the area which will be afforded the highest priority protection.

However the remarkably consistent long term trend in bank line movement over this reach as a whole suggests strongly that the current quiescent conditions immediately to the north of Sariakandi may be expected to end within the coming five to ten years. This could well be replaced by a period of active erosion, possibly centred about 3 to 4 km north of Sariakandi. Such erosion would then threaten the flank of Kalitola Groyne and necessitate the construction of further bank stabilization works to prevent a possible breakthrough into the Bangali in this area. It is even possible that the changing focus of erosion will take place sooner and that the priority for stabilization will shift from Mathurapara to north of Sariakandi. Close monitoring of the situation is thus called for, and further measures north of Sariakandi will be required in the short term.

50

4.5.2 Layout of the Works

Two standard hard-point structures will be required as shown in Figure 2.4, one approximately 3 km north of Kalitola Groyne, and the other approximately 8 km north of the groyne, east of the village of Naodabaga. The hard-points will be linked back to the BRE by means of cross-bars.

4.6 **Fulcharighat**

4.6.1 The Design Concept

The December 1990 aerial photography shows that the small radius aggressive bend that had been active until 1989 had reached its peak and stalled. In its place a large radius long wavelength bend was developing, scaled on the large bend at Gabargaon on the left bank. From the limited SPOT coverage available it is possible to follow the evolution of the Gabargaon bend. Between March 1989 and November 1990 it had moved laterally almost 1,000 m, reached a radius to width ratio of about 6.0, and was showing all the signs of stalling. The final planform corresponds closely with the set of maximum large bend planforms that have been catalogued under the BRTS morphological studies. Superimposing this bend evolution onto the Fulcharighat 1989 bend leads to the inference that the latter could develop rapidly to the form shown on Figure 2.6 and soon after that it would most probably stall and a cutoff would develop.

In terms of the security of the BRE, it seems probable that it will be breached north of Fulchari but if the bend migrates downstream then the breach could occur in the vicinity of where the railway line crosses the BRE. In either case flooding of the permanent railway facilities west of the BRE would be severely affected.

From inspection of the anabranch pattern evolution over recent years it seems that there are two principal planforms that can develop, one based on the prevalent shorter wavelength thalweg meander and the other on the large bend form that is currently dominant. This is reinforced by the presence of the small metastable char just north of Fulcharighat which has all the attributes of a secondary level metastable char. The secondary chars further downstream are less well defined which may be due to the fact that they are still evolving but may also be because they tend to be subjugated by the change from short to long wavelength dominance.

After the large bend reaches its apogee and stalls the most likely development is that the resulting cut-off channel will re-establish the shorter waveform, the exact wavelength of which will depend on the annual sequence of flood magnitudes at the time but may be expected to fall within the range 15 to 17 km.

Taking a medium to long term view therefore, any bank stabilization measure should be such that it can accommodate these two waveforms. An embayment corresponding to the shorter wavelength is a strong persistent feature over recent years and also appears on older 1:50,000 maps. It would therefore seem to be an appropriate first stage stabilization target.

S1

4.6.2 Layout of the Works

Two hard-point revetment structures are proposed, each with 300 m straight revetment. They would be sited one approximately 2 km north of the railway line, the other approximately 2 km south, and linked back to he BRE by means of cross-bars.

4.7 Kazipur

4.7.1 The Design Concept

Kazipur lies opposite Island D (Figure 4.1). It was the scene of unusually severe bank erosion between 1988 and 1990, associated with one of the shorter wavelength aggressive bends which reached a peak in 1990. The deep scour trench that was created moved about 1 km downstream during the 1991 monsoon season with reduced rate of erosion. This bend now appears almost to have completed its life cycle.

It has been noted that the cross-over between Islands D and E is poorly defined to the extent that in recent years the flow has only been from east to west and the two islands have tended to overlap and are now near to merging. If this trend continues, the two islands may coalesce to form one island of about 36 km length. This implies a major anabranch wavelength of about 72 km or around double that of the theoretical single thread wavelength for dominant discharge, the nearest equivalent of a single anabranch bend of this scale being the 33 km long reach to the east of the current extended Island B. From the time snapshots of the river available it may be inferred that such a bend is not a long term stable feature, in which case there are two principal modes of evolution:

(a) the island might become attached to left bank;

(b) the two anabranches might develop individually on either side of the island to form a multiple channel anastomosed system. The feature north of the Dudhkumar river, which has a major island length of about 36 km, may represent such an evolution.

In either case the right bank would come under increased attack, with the former probably creating the more aggressive condition. An average bank retreat rate of over 100 m/y over the reach could be expected with the upper part between Kazipur and Simla, where the current anabranch width is only about 3.5 km, experiencing more attack than the lower wider section.

For reference: the maximum widths of the "Dudhkumar" feature scaled off the 1765, 1830 and 1988 maps were 6, 12 and 15 km respectively, giving averaged widening rates of 92 and 19 m/y and a long term average of 40 m/y.

4.7.2 Layout of the Works

The layout of the works in shown in Figure 2.7, with two standard hard-point structures (300 m straight revetment) approximately 5 km apart, connected to the BRE by cross-bars.

52

4.8 Betil

4.8.1 The Design Concept

It has long been recognised that the pattern of the river south of Sirajganj is significantly different to that to the north. The more detailed analysis of planform characteristics has merely emphasized this without throwing up any clear explanation for the reason behind it. The most significant parameter change is perhaps the water surface slope that shows a distinct flattening south of Sirajganj. This is accentuated by the superimposed backwater effect of the Ganges (see First Interim Report).

Compared to Sirajganj, where the net westward movement of the bankline has been limited to about 1 km since 1830, this lower section of the river has experienced major bankline movements and the right bank is now some 5 km further west than it was in 1914. All the indications are that the river will continue to move westward in this reach, both through widening of the anabranch and an overall drift, at an average rate of about 100 m/y.

Although there are signs that the river has strong tendencies to maintain a meandering planform, it has been unable to settle down to any regular meander pattern. This may be attributable to the speed with which the relatively short wavelength high sinuosity large bends evolve and die through cut-off development. These cut-offs then tend to evolve into braid bar islands but find difficulty in taking up a regular pattern of their own, perhaps because of the widely ranging flows that they carry.

The west anabranch flanking Island F is a case in point. It has no clearly defined evolution of secondary metastable chars and the thalweg planform displays a very irregular waveform. Its future is closely linked to the development of the "throat" section upstream, which only 30 years ago was an island reach and unless the JMB is constructed in the near future it may revert to that state. If that were to occur then instead of the unstable and unequal split of flows around Island F, there would be a rather more stable cross-over condition.

With the construction of the JMB, there is a high probability that a char group will develop a short distance downstream of the bridge where the mean velocity reduces, reinforced by the Ganges backwater effect. This is likely to have a major impact on the pattern of char development, which is largely unpredictable at this stage.

As a short-term measure for bank stabilization, there seems little option but to place a series of hard-points along the right bank in the anticipation that this will encourage the evolution of a more regular pattern. This lower reach is dominated by the larger waveform bend and there is a dearth of shorter wavelength bend information on which to base an alternative plan. In the absence of such natural tendencies it is reasonable to expect that the introduction of a regular spacing of hardpoints will in itself establish a pattern.

4.8.2 Location of the Works

The first two hard-points to be constructed on this reach under the Master Plan will be those shown as Phase 1C works in Figure 2.8. They will be standard hard-point structures with 300 m long straight revetment sections, linked back to the BRE by means of cross-bars. The

53

locations are planned to give a measure of protection to the town and the handloom industry, as well as to the retired BRE.

4.9 Note on the Locations of Phase 1 Structures

Locations of each of the Phase 1 structures have been described in the preceeding sections of this chapter, and illustrated in the figures referred to. It is important to realize, however, that the programme for construction even of these "priority" structures is likely to take fifteen to twenty years to implement.

With the exception of the works at Sirajganj and Kalitola Groyne, both Phase 1A for early implementation under the River Bank Protection Project and both augmentation of existing protection measures, the river bank may have moved very considerably from the positions shown in the figures, which, for the Phase 1B and 1C works, form the basis of the tender drawings in Annex 6. It is essential for all these sites, and this includes the hard-point locations at Sariakandi and Mathurapara in Phase 1A, that detailed surveys be carried out as near as possible to the time of construction.

Such surveys will permit the locations of structures, and retirement of associated lengths of the BRE, to be optimized in the light of the most recent developments in the planform of the river bank. The interval between survey and construction should be the minimum that will permit the due process of land acquisition, and the execution of any design modifications which may be necessitated by the altered circumstances.

54

5. CONSTRUCTION METHODOLOGY

5.1 Form and Scope of Contracts

Work at the priority locations falls into the number of distinct parcels of activities:

i) BRE

Planned Strategic Retirement of the BRE embankment before it reaches a state where there is a high probability of breaching as a direct consequence of river bank erosion. This may include realigning parts of the recently retired embankment between Sariakandi and Chandanbaisa as an integral part of the arrangements for minimizing the risk of a breakthrough by the Brahmaputra into the Bangali River, and similar realignments at other priority locations.

The work is of a type and scale that is suitable for execution through the normal BWDB Local Competitive Bidding (LCB) arrangements, based on contract documentation and procedures that have been approved by the IDA (World Bank). It is important however to ensure that the works are carried out to the full specification, including adequate compaction of the fill material. Consideration should be given to the independent monitoring of compliance with the construction methodology procedures, particularly with respect to the proper testing of materials and in-situ densities achieved.

ii) Phase 1A

a) Upgrading the bank protection measures at Sirajganj and extending these to link into the existing Ranigram Groyne, the nose of which will also be strengthened to form an upstream termination for the revetment. This work forms the first phase of a strategy for the longer term security of Sirajganj and, in due course, the proposed Jamuna Bridge. The existing revetment is in imminent risk of large scale failure since it does not have any functional filter below the armour and in many places the bank slope is excessively steep. It has been concluded that the most cost effective approach is to reconstruct the revetment to an appropriate design specification, including the provision of an adequate falling apron at the toe; as much as possible of the existing concrete block armour material will be reused.

b) Construction of Hard-Points near Sariakandi and Mathurapara, as the first phase of a strategy to minimise the risk of the Brahmaputra breaking through into the Bangali River. The work will consist of the construction of two lengths of bank revetment of nominal lengths of 300 m each. These hard-points will be linked to the main BRE embankment by cross-bar embankments, whose function, in combination with the substantial upstream and downstream revetment terminations, is to guard against outflanking of the structures when intervening embayments occur.

In addition to the two hard-points, the existing Kalitola Groyne will be upgraded to act as the upstream anchor for the stabilized reach and in addition to provide a more reliable level of protection from erosion for the township of Sariakandi. This groyne has similar design deficiencies to those mentioned in relation to the Sirajganj revetment.

iii) Phase 1B

a) Construction of Hard-points at Simla and Sailabari Groyne to secure the reach north of Sirajganj and thus prevent any outflanking tendency which would otherwise threaten Sirajganj town. One hard-point with a 300 m long straight revetment section and upstream and downstream terminations will be provided at each location, and linked by cross-bar to the BRE.

b) Construction of Hard-points near Naodabaga and Sariakandi North to secure the reach north of Sariakandi. These hard-points will perform a similar function to those described above in preventing outflanking of Kalitola Groyne and the consequent loss of Sariakandi. They will also be crucial in preventing breakthrough of the Brahmaputra into the Bangali, where the rivers run as close to each other as they do further south.

iv) Phase 1C

a) Construction of Two Hard-points at Fulcharighat to provide security to the town and railhead. These will be similar structures to those described above, connected back to the BRE.

b) Construction of Two Hard-points at Kazipur to provide security to the town and the adjacent reach of river. The hard-points will be similar to those described above, connected by cross-bar to the BRE.

c) Construction of Two Hard-points at Betil to provide security to the town and the hand-loom industry there. The hard-points will be similar to those described above, and connected to the BRE.

The civil engineering works described under ii), iii) and iv) are of a similar nature and will involve the same type of expertise, construction management capability and resources. Sirajganj, however is unique in that it comprises a continuous length of revetment and work in the river will span two dry seasons. The remaining river training parcels (or contracts) all include two hard-points to be constructed during a single dry season in the river, and Sariakandi/Mathurapara includes within its scope of works the reconstruction of Kalitola Groyne. Contracts to be let simultaneously, e.g. the Phase 1A works, could feasibly be awarded to one contractor (or joint venture) provided that he had adequate resources and management strength to operate efficiently at the separate locations concurrently.

The period available for riverside working will be limited to six months per year at most and for part of this time the conditions created by the rising or falling river levels will create a very difficult working environment. Perhaps the most demanding task from the point of view of quality control will be the accurate placing of geotextile and armour material in relatively deep flowing water.

This work will require a higher level of construction organisation and management and the ability to rapidly mobilise the correct plant and equipment than is currently available in

56

Bangladesh. It can therefore only be satisfactorily carried out by prequalified international contractors, with the contract management and quality assurance monitoring entrusted to an equally experienced international organisation. If the financing is to be provided by one of the large international agencies, then the selection of the contractor will be through approved International Competitive Bidding (ICB) procedures and the form of contract will be FIDIC or an equivalent.

5.2 Principal Constraints on Construction Programming

5.2.1 Appointment of Contractor

The ICB procedures for the appointment of contractors for these major civil engineering works cannot be completed in less than eight months, under the most favourable conditions, and more usually takes around 12 months. Time must then be allowed for the contractor's mobilisation and temporary land acquisition (four months), setting up his facilities (four months) and the procurement and stockpiling of materials (minimum four months for precasting concrete blocks and possibly less for imported rock).

If the contract award can be made awarded prior to October in any year then this timetable would permit the commencement of construction at the start of the subsequent dry season. An earlier award would have no particular advantage other than allowing the contractor more time in which to negotiate for temporary land acquisition, construct accommodation support facilities, commence stockpiling armour material and generally to get organized for the intensive activity during the six month construction seasons. It must however be made clear that possession of the site, with its implications regarding permanent land acquisition by the Employer, is not guaranteed until a realistically attainable date.

5.2.2 River Conditions

The major part of the works, both financially and physically, are to be constructed below mean LWL, much of it more than 8 m below. Since dewatering on this scale is likely to be impracticable, this implies that the majority of the works will have to be implemented underwater.

The river level begins to rise in April of each year and typically climbs quite rapidly to reach bankfull towards the end of June or early July. The monsoon rains in northern India keep the level up during July and August and it begins to fall again during September. During these six months navigation by larger vessels is possible, although the shifting channels mean that constant vigilance is called for and particular care must be taken during the falling stage when sandbars are emerging. Working underwater however becomes very difficult not only because of the poor visibility but also due to the relatively high flow velocities, turbulence and rapid sediment transport. Temporary current deflectors and coffer dams are liable to be undermined or simply washed away. Placing geotextile mats and armour material to line and level therefore becomes a very hazardous and probably impracticable task no matter how sophisticated the equipment utilized.

During the six months of lower flows conditions are most favourable for underwater work for 4 to 5 months but with good organisation and management and careful planning, productive activity should be possible for the full six months in most years. Even at low flows the

6X

sediment levels in the river make visibility under water poor and divers will be obliged to work largely by touch. Navigation at this time of the year is liable to be difficult for larger conventional vessels and considerable dredging may be necessary in order to maintain regular direct water access to the works. The Contractor will therefore have to adopt a flexible approach to transport with the siting of wharfs decided only as the river levels begin to fall and the channel patterns and char locations begin to become more firm.

5.2.3 Periods to be Allowed for Partial and Full Completion

There are significant hazards associated with completing only part of the works in one season which have to be offset against the logistical implications of completing all major works within a narrow six month window.

In the case of Sirajganj it would, for example, be possible to carry out the upgrading of the existing 1.5 km of revetment in the first season with a temporary upstream termination, and then to complete the extension to Ranigram Groyne in the second season. The risk would arise if a large flood occurred during the intervening season and the groyne came under serious attack. This risk could be largely offset, at a cost, by ensuring that materials, plant and equipment were to hand for emergency protective works. It would also be feasible to reverse the order of phased completion, in which case the risk during the intervening year would be that failure of the existing revetment might occur. The choice between these two options would depend mainly on the thalweg planform, and therefore the likely focus of erosive attack at the time. It will, however, be necessary to define the sequence prior to tender from the point of view of handing over portions of the site to the Contractor, and defining partial completion dates. The present tender documents require the upstream section completed first. The problem is particular to Sirajganj.

The choice between one or two season construction will depend mainly on the balance between:

one season: low risk of damage during construction; no down-time for plant and equipment and only one mobilisation/demobilization (although possibly approaching twice the plant and equipment); high risk of failing to complete in the event of unforseen problems; major cost to the Employer if the Contractor is able to claim that failure to complete was due to conditions outside his control (e.g civil disobedience, exceptional weather, late possession of Site); lower establishment costs for both Employer and Contractor.

two season: high risk of controllable damage to existing protection or new works during intervening monsoon season, low but finite risk of uncontrollable damage; two mobilization/demobilizations or substantial downtime for plant and equipment; reduced opportunity for major contractor claims; higher establishment costs for both Employer and Contractor.

It is apparent that the one season option offers a potentially lower cost of construction if the Contractor has the resources and expertise to be confident of completing the task. The risk of major over-runs will however be borne jointly by the Employer and the Contractor (the ratio depending on the proportion they may be assigned to causes outside the Contractor's control). Under the second option the minimum cost must be higher because of the

S8

provisions that have to be made to cover the risks involved during the intervening season and the relatively lower utilization rates for the plant and equipment (this may be minimized by using water transport, and the like for building up stockpiles of armour material during the monsoon season). The risk of unpredictable cost increases will be largely associated with the possibility of significant damage occurring during the intervening season; only if these are related to inadequate provision made by the Contractor will they be assignable to him, the cost of other remedial work will have to be borne by the Employer.

In view of the relatively hazardous and unpredictable environment in which the works are to be carried out, the minimization of the duration of exposure would appear attractive. On the other hand such an approach does leave the way open for an experienced contractor to establish very large claims; in theory, a one day delay due to matters outside his control (e.g a problem over possession of site) could provide the basis for a claim based on a more than doubling of his fixed charges (i.e a switch from one to two season construction).

The tender documents for Sirajganj allow the Contractor two full seasons in which to complete the work. As noted earlier, the timing of award is important and the precise contract period should be determined only once there is a clear perception of when award will be.

The situation at Sariakandi/Mathurapara is similar but the risks of adverse conditions developing during the intervening season are significantly greater and the steadily decreasing distance between the Brahmaputra and the Bangali increases the urgency of intervention. Based on the present pattern of erosion, the minimum requirement would be to construct the Sariakandi Hard Point and to complete the upgrade of Kalitola Groyne during the first season, leaving the Mathurapara Hard Point for the second season. Unlike the Sirajganj situation, there would be little that could be done to counter any on-set of rapid bank erosion that threatened to result in a breakthrough to the Bangali River. In the event of such a breakthrough occurring, re-establishment of the bankline and the BRE would probably necessitate the construction of a groyne/hard point in deep water and exposed to the main river flow, which would become a far more difficult and costly exercise.

In this case the balance is more in favour of a single season construction, with work proceeding on at least two sites concurrently. Liquidated damages applicable in the event of the Contractor's failure to complete on time should be made realistically high to cover the risks involved.

For the remaining parcels under Phase 1, similar arguments to those for Sariakandi and Mathurapara apply, and the tender documents are drafted on the basis of one season working in the river.

5.3 Availability of Materials

The sources and availability of the principal materials required for the works have been described in Annex 5 of the BRTS Second Interim Report. In accordance with the requirements of the BRTS Terms of Reference, the design of the works has taken into account the Government's preference for the use of local materials, labour and other resources, as far possible. Consideration has also been given to the material requirements for maintenance of the works that will arise following demobilization of the main contractor.

59

The importation of some materials cannot be avoided and in the case of others there may be financial and environmental arguments that favour this. The major material requirements are:

Cement

The local cement supplies are insufficiently reliable for a critically time dependent job such as this and it will have to be imported for this reason.

Geotextile

The use of a reliable filter is an essential requirement for a sustainable revetment design and geotextile is at present the only practicable material for this purpose.

There is no geotextile manufactured in the country at present, although the scale of works on this and other projects anticipated in the coming five years that will require geotextiles may encourage international manufacturers to set up facilities in the country. The raw materials would still have to be imported.

Concrete Blocks

Although the choice between rock and concrete blocks for the armour layer below LWL+2 will finally depend on the respective prices quoted by the successful contractor, blocks will be used above this level.

Of the ingredients, the formwork, sand and coarse aggregate are all available from sources within Bangladesh and the Brahmaputra water is suitable for concrete. The use of brick aggregate rather than crushed stone is expected to result in lower overall cost, but the impact of the strict environmental management controls that will be imposed on the contractor and the effect of the relatively high demand on the market price will not be known until tenders are received and the options are being left open. Some coarse aggregate stone is imported from India and some is available from within the country at Sylhet. As mentioned above, it has to be assumed that the cement will have to be imported. Quarried rock, if used, will have to be imported, most probably from India.

Sand Bags

Jute sand bags are readily available in the country and are relatively inexpensive. They are suitable for certain forms of temporary works and permanent bulk fill but their low strength is a significant limitation in many cases and they have no filter properties. As temporary works at the Contractor's option, they are not specified although proposals will require the Engineer's approval.

The use of imported geotextile bags is necessary in situations where a filter function is required and where the strength, flexibility and durability of geotextile is needed for situations such as multiple reuse, handling by mechanical equipment, and resistance to puncturing/tearing.

5.4 Land and Water Access

5.4.1 Water Access

Since all the works are situated along the river bank, the use of water transport is clearly indicated. The principle limitation will be the unpredictable shifting of naturally navigable channels during high flow periods and the associated appearance of massive sand bars as the river level falls.

By their nature as locations of priority for bank protection, both Sirajganj and Sariakandi have at present persistent large channels in the close vicinity of the right bank and although it is not expected that this situation will change significantly before the start of construction, it is not possible to guarantee this. Of the two locations, there is a higher probability of major changes in the braid pattern in the vicinity of Sariakandi than near Sirajganj where the macro anabranch pattern has been relatively stable over at least the past 30 years. At the remaining Phase 1 sites, with their relatively later scheduling for construction, the probability of major changes is that much greater.

It is reasonable, on the basis of currently available morphological evidence, to expect even in the case of Sariakandi that a deep water channel will be contiguous to the right bank somewhere within the 6 km length between Kalitola Groyne and the site for the Mathurapara hard-point and that there will be a navigable route from here to both Fulcharighat and Sirajganj at all times of the year, although careful navigation will be required at low water levels. The deep water to the south of Sirajganj is expected to persist at least for long enough to see the Phase 1A works complete.

Both Fulcharighat and Bahadurabad offer good potential interfaces between rail and water transport, although the Contractor will have to be negotiate his own arrangements with Bangladesh Railways with regard to the means of transfer. The former is better located for materials such as rock coming from India while the latter has better facilities.

There are good all year round road/water interfaces maintained by BIWTA at Aricha, Nagabari, Sirajganj and deteriorating conditions at Bhuapur. There is probably also potential for making use of the facilities at Bera.

Thus the use of water transport for the movement of the bulk materials is clearly not only feasible but the most practicable mode for most locations. It is envisaged that the Contractor will establish a jetty/wharf at each site at a location where there is a good probability of the channel remaining navigable for the duration of the job and to link this by road to his main storage and work areas for stockpiling in the non-construction season. If rock is the selected armour material, it may prove possible to ship this direct from an Indian river port in order to reduce transhipment costs.

Transport of materials from the storage areas could be either by water or road depending on the Contractor's preferred method of working.

61

5.4.2 Land Access

The main national road system is suitable for the transport of the materials required for the job but the access from the main roads to the river bank is generally poor, the best on the right bank being at Sirajganj.

There is a well constructed asphalt surfaced road from Bogra to Sariakandi but it is not designed for large amounts of heavy traffic and would probably deteriorate seriously over the course of a construction season, and the only means at present of crossing the Bangali is by ferry. Use of this road by the Contractor should therefore be restricted to light vehicles and a very limited number of heavy vehicles per day. Clauses in the civil works contract requiring the Contractor to maintain any roads that he uses, and upgrade them if necessary (including ferry facilities) will be strictly enforced. Similar problems of land access exist at most of the remaining Phase 1 sites.

The Contractor will be responsible for making his own arrangements for land access from the national road system to his accommodation and work sites. It is envisaged that for this purpose he will opt for temporary roads with brick paving of a type that is common in Bangladesh (e.g the temporary roads for ferry ghats); these will facilitate reinstatement of the land on completion of the works. He will be specifically prohibited from using the BRE for access unless he can come to a separate agreement with the BWDB that will ensure both that its function is assured and that squatter settlements of displaced persons are not disturbed.

At Sirajganj the situation is better but vehicle access through the centre of town is very congested and this route is not suitable for the haulage of bulk materials. In fact it is proposed to ban all Contractor's traffic from the centre of town and to restrict him to the peripheral roads, which choice he would almost certainly make for himself in any case. Use of the feeder road from the main Nagarbari to Bogra road will be restricted in terms of maximum axle loading.

5.5 Transport of Materials

The Contractor will be entirely responsible for making his own arrangements for transport, subject to the limitations set out in the previous section. This will require careful planning and close liaison with the respective Government organizations in order to ensure that a smooth flow can be maintained. Water transport is seen as the most appropriate mode, particularly in respect to environmental considerations, with as much use as possible being made of the railway system for the movement of bulk materials. However total dependence on water and rail would not be practicable and limited use of the road system must be permitted.

5.6 Contractor's Yards and Facilities

The Contractor will be entirely responsible for making his own arrangements for his and the Engineer's accommodation and for all his casting yards, workshops, storage areas and wharfs. This will extend to the procurement of the land, provision of services, maintenance and removal on completion of the works. Siting of these areas will be subject to the Engineer's approval, which will not be unreasonably withheld, and in accordance with government regulations.

62

In the case of Sirajganj the Contractor will be offered temporary use of part of the area to be reclaimed between Ranigram Groyne and the main town.

5.7 Construction Activities

5.7.1 Common Features

Although the sites of Sirajganj and Sariakandi/Mathurapara have their distinctive characteristics that will influence the method of construction, they are similar in most important respects to the remaining Phase 1 sites and the timing and form of activities will be broadly similar. The principal difference is that at Sirajganj works in the river will proceed for two dry seasons. Possible construction programmes are shown in Figures 5.1, 5.2 and 5.3.

Construction Timetable - Preliminary Works

For this purpose it is assumed that the Letter of Acceptance will not be issued before June in the relevant year. The notice to commence may follow a month later when contract conditions such as the provision of bonds have been satisfied. The first three months will be occupied by the Contractor organising the mobilization of his plant and equipment, identifying sources of supply for materials, arranging sub-contracts, and acquiring land for accommodation and temporary works. This will take him through the main monsoon season of July/August when land-based work would be difficult and he will be able to commence work on site in the drier conditions which will follow.

At Sirajganj, hydraulic fill for reclamation (e.g.of part of the area between Ranigram and the existing town protection) can commence as soon as the Contractor can mobilise a dredger for this purpose (provided that land acquisition/wayleave arrangements can be completed in time).

The critical path activities during the first part of the various contracts will be:

(a) acquisition of land for storage, accommodation, and other temporary works;

(b) carrying out bathymetric surveys to allow the layout of the permanent works to be finalized by the Engineer;

(c) the manufacture of brick aggregate concrete blocks, presuming that these prove to be the lowest cost solution offered by the successful Contractor.

During the dry season late in Year 1 and early in Year 2 the Contractor will construct his site accommodation and his block making facilities and/or commence stockpiling rock and other essential materials.

All preliminary temporary works will be completed before the start of the monsoon season of Year 2 and stockpiling of armour material will continue through the monsoon so that at the start of the following dry season construction of the permanent works can commence without delay.

63

Bathymetric surveys will be carried out during both the Year 1 and Year 2 monsoon periods and the intervening dry season in order to build up a better picture of channel evolution and to provide the basis for the finalization of the layout of the works.

As the river level starts to fall in September/October of Year 2 the Contractor will initiate his main permanent works construction programme.

Construction Methods

Slope Preparation

(a) in relatively tranquil areas where the water depth is less than 8m below LWL

Excavation of the trench that will take the apron may be the first action, particularly where there will be an excess of dredged material over local fill requirements. The dredged material may be dumped over the section of apron that has been completed upstream in order to discourage the formation of a deepwater channel leading into the current working zone, or placed strategically to provide temporary diversion of flow away from future work areas. This will have to be carefully controlled and carried out in such a way as not to induce adverse effects further downstream or on the banks of the metastable islands (or even to be perceived to have such consequences).

Where the new slope line requires that material be excavated/dredged from below the LWL, then this will be carried out by either pontoon mounted or landbased excavators of the backhoe or dragline type, or by cutter suction dredger. Material will either be placed locally to form hydraulic fill to the approximate slope line as required either above or below LWL, pumped to a stockpile, or moved away by barge for reclamation work elsewhere. Excavators will be used for this purpose where there is any likelihood of encountering under water obstructions.

Where the new slope line requires only fill below the LWL then this will be provided from the material dredged from the outer part of the apron trench or imported from another part of the works. It is assumed that the fill so placed will have a riverside slope of not flatter than 1V:10H. Once the fill has reached the current water level, which may be 2m or more above LWL for much of the construction season, then containment bunds will be formed either by bulldozer or by dumping from a backhoe/dragline depending on the working method favoured by the Contractor. The outer slope of the bunds will approximate to the finished slope line, being if anything proud of this line. These containment bunds must be well watered to ensure full consolidation. The ponds so formed can then be filled by pumping in sand. The flatter than required slope below water level will be trimmed back by some form of dredger shortly prior to the placing of the apron, the underwater geotextile and the underwater slope armour. Finally the upper slope will be trimmed by dragline, backhoe or other mechanical means (or possibly even by hand) immediately prior to the placing of the upper slope protection. Particular care will be required to ensure that the interface between the upper and lower slope protection is properly formed.

(b) in deeper water, but normally not exceeding 14m below LWL, with higher than average velocities. Some scour of unprotected river bank will occur even at low water.

The procedure would be much the same as (a) but the Contractor may be obliged to improve working conditions by taking measures to divert the high velocity flow away from the working area by dumping of "unsuitable" salvaged material or reusable geotextile bags to form temporary underwater groynes and diversion dams and/or by dredging. Such temporary works will have to be approved in advance by the Engineer.

By the nature of these conditions, the existing underwater bank slope is likely to be considerably steeper than required for the finished section and extreme caution will have to be taken over excavation below water level that could lead to erosion beyond the required finished line. Where the design allows for cutting back the existing slope then this will be a relatively straightforward operation that can be either carried out from pontoons or the bank, as described under (b).

Where underwater fill is required to form the slope profile and the velocity is too high for conventional hydraulic fill then the profile may be built up by the careful dumping of sandfilled jute bags. If this operation is well planned, the bags can be laid to form underwater groynes that will keep the high velocity flow away from the bank and may even permit hydraulic fill infill. The Contractor will have to take care to ensure that the bags do not burst during placing and that they form a well consolidated mass. Bags must not be overfilled and it may be better to lash then into bundles and mats to be placed by crane or hydraulic arm. The finished surface must be sufficiently smooth to ensure that the geotextile membrane does not "tent" across voids. Coarse granular fill (e.g. river gravel) may be used for this purpose. Diver inspection will be used to check that the surface is suitable before the placement of the geotextile.

In order to give the Contractor as much flexibility as possible, payment will be made for the fill to the finished lines, irrespective of which method the Contractor opts to follow. No payment will be made for any additional fill that may result but equally the Contractor will not be required to remove material that is outside the design profile provided that this does not detract from the performance of the finished works and is consistent with the intent of the Specification. Filling of local depressions with armour material will be permitted but no additional payment will be allowed for this.

Placement of Geotextile and Armour Material

In some places the existing bed level may be deeper than the nominal apron setting level. In such situations the apron material will be dumped straight onto the bed in such a manner as to ensure that it will deform satisfactorily. Physical model tests have indicated that deformation performance of the apron is not sensitive to its geometry - the main condition to be met is that the required volume of material is provided. The apron should contain adequate material to ensure not less than two layers of armour on the scoured slope after allowing for adverse distribution.

The order of construction will have to be (1) place the geotextile on the lower slope (i.e below water level) together with its ballast layer, (2) dump at least part of the apron material to anchor the bottom of the geotextile, which extends 2 m under the apron for this purpose,

65

(3) place armour material on the lower slope, (4) complete the placing of the apron. It may be that part of the slope armour placing may be carried out by land based equipment but the apron material will almost certainly have to be dumped from pontoon/barge. Placing of the slope material will require a considerably higher standard of control than for the apron material.

Crest and Access Road

Where fill is required to form the revetment and the Contractor chooses to place the fill hydraulically, consolidation will occur swiftly. The material will be predominantly sandy, and the Specification requires that measures be taken to avoid the inclusion of silt. The fill will therefore be relatively free draining and will quickly become trafficable for further construction activities. BIWTA have found that sandfill placed within containment bunds a metre or so above water level took 4 to 6 weeks to dry out, depending on the grading of the material. Conditions under which fill will be placed in the revetments will be appreciably more favourable, and drying out commensurately faster.

If fill is transported by truck, for instance from a stockpile of dredged material on land, it will, above water level, be compacted in layers as specified. The effects of settlement will therefore be minimized and trafficking will not pose a serious problem.

The design includes a crest retaining wall and access road, and in view of the unsightly appearance which would result were settlement to occur, construction should be left as long as practicable after filling. It is planned that settlement be monitored and that the crest works be completed only when it has reduced to an acceptable level.

The crest wall is designed as an in situ concrete retaining wall, although the Contractor is permitted to propose precasting it subject to the Engineer's approval of method. The crest access road will be of bitumen macadam at Sirajganj, where it will be heavily trafficked, and herring bone bond brick will be used at the hard-points elsewhere.

5.7.2 Particular Conditions at Sirajganj

As described in detail in Section 4.2.2, the Works will consist of the following main components:

- Section A - Upstream termination incorporating upgrading of the nose of Ranigram Groyne.
- Section B - Land reclamation behind Section C over an area of about 25 ha.
- Section C - Length of new revetment spanning between Ranigram Groyne head and the upstream limit of the existing town protection revetment, a distance of about 700 m.
- Section D - Replacement of the existing revetment over a distance of about 1,500 m. and a downstream termination.

GG

The overall bank slope along the full length of the existing revetment is potentially unstable under even modest earthquake loading. Any additional scour is likely to result in local collapse of the type that resulted in the Jailkhanna embayment, which could rapidly develop into a major progressive erosive failure. None of the existing revetment has an effective filter layer under the armour and as such is vulnerable to soil migration failure induced by high velocity river flow or seepage flow in the bank during the falling stage.

The unpredictability of the river morphology means that at the time of construction any part of the site may be experiencing attack. There is however only a very low probability that deep scour of the type that developed at Kazipur in 1990-91 will occur at Sirajganj during the construction period (i.e up to 17 m below LWL). It is however likely that there will be localized scour to a maximum depth of around 10-12 m below LWL which may be associated with higher velocities up to twice the mean low flow velocity; these zones may be of the order of 200-400 m long. It is these deeper troughs that will present the principal difficulties during construction.

<u>Construction Programme - Permanent Works</u>

After completion of the preliminary works activities, when the river level starts to fall in September or October 1995, construction of the Permanent Works will start with the following critical path activities:

(a) commence the salvaging of the existing armour blocks from the nose of Ranigram Groyne (or whichever other part of the works may be deemed to have priority at the time);

(b) the Contractor will submit for approval his detailed proposals for temporary works and dredging for the permanent works including full hydraulic fill/disposal details for the approval of the Engineer;

(c) commence dredging and place fill for the core of the upstream termination.

By the end of the low flow season (April/May 1996) the whole of Section C must be complete and preferably also part of the new revetment up to about the junction of the road leading into Sirajganj Town (grid 6100) making a total completed length of about 900 m. At this point the revetment may be temporarily stopped by wrapping it a short distance back into the bank and dumping a large mass of armour material to blend into the existing slope protection.

By early the following monsoon season (June to mid-October 1996), the Contractor should have completed hydraulic filling in Section B. Otherwise the principle activity will be stockpiling armour material ready for the final construction season. The high river stage will facilitate the importation of materials by water, particularly if it is being brought downstream.

The final dry season (1996/97) will see the construction of the second part of Section D, following the sequence of activities outlined above. Removal of temporary works and clearing up of the Site will have to be completed by no later than the end of June 1997 in order to be clear before the main monsoon period.

The total duration of the Contract, if started in June/July 1994, would be 36 months.

Construction Method

Recovery of Concrete Blocks for Reuse

(a) in relatively tranquil areas where the water depth is less than 8m below LWL.

Existing armour blocks above LWL will be recovered either by hand labour using local sub-contractors or by hydraulic backhoe and either stockpiled in areas on the landward side of the flood embankment/road for future use or carried straight by barge for dumping as apron material or for temporary flow diversions/scour hole fill. Pontoon mounted face shovels or backhoes could also be used for this purpose and would be more efficient if the material is going to be immediately reused. The recovery operation will not be allowed to advance further ahead of reconstruction than is safe taking into account the time of the year and the local conditions below water level.

(b) in deeper water, but normally not exceeding 14m below LWL, with higher than average velocities. Some scour of unprotected river bank will occur even at low water.

The procedure would be much the same as (a) but the Contractor may be obliged to improve working conditions by taking measures to divert the high velocity flow away from the working area by dumping of "unsuitable" salvaged material or reusable geotextile bags to form temporary underwater groynes and diversion dams and/or by dredging. Such temporary works will have to be approved in advance by the Engineer.

Reuse of Salvaged Blocks

Salvaged armour material may be reused in either permanent or temporary work according to its condition. The Engineer will determine whether the salvaged material is suitable for the permanent works and where it will be used; only suitable material will be measured for payment as armour material, other material may be included as fill in locations approved by the Engineer or used by the Contractor at his own expense for temporary work. It is envisaged that the armour material would be recovered on the barge by a sluicing process. The sand residue would then be pumped ashore as hydraulic fill wherever required.

The existing concrete blocks along the revetment at Sirajganj are of variable condition. Some are in very poor condition indeed with extensive evidence of poor quality control - lack of vibration, poor aggregate grading, low cement content, etc. These blocks generally have a rounded appearance, exposed brick aggregate and friable surface. Other blocks, the majority overall, are quite acceptable for reuse. The blocks are most commonly 18 in size, i.e. smaller than the design size for the new revetments. Those suitable for reuse, however, can be used in non-critical areas such as the apron of the downstream termination. Blocks rejected for reuse can be dumped on the river bed within the area of filling for the new embankment.

Upgrading the Nose of Ranigram Groyne

The sequence of construction will be largely as described above, with the depth of water depending on the main thalweg course at the time and the portion of the termination. Blocks recovered from above water level may be used to form the base of the termination fill below water level if the water is deep and the flow velocities relatively high. Alternatively the sandbag fill approach may be followed to form a coffer dam within which hydraulic fill may be placed. Above water level the fill procedure should be the same as described above.

The new revetment between Ranigram Groyne and the existing town bank protection.

The only difference between this section and the standard revetment described earlier lies in the formation of the artificial fill profile that will support the armour layer and form the containment for the landfill behind.

Much will depend on the behaviour of the thalweg at the time of construction. A significant branch of the main channel cut right across this section during 1992. If this were to be the situation at the start of construction, then serious consideration will first be given to temporarily diverting it away from the work area. If this is impracticable and realignment of this section is not feasible then it will be necessary to treat the core of the revetment much as one would a harbour mole or breakwater. Either sandbags or sand asphalt would be used to form the river side containment bund/core. The landward face of the core would then be constructed by hydraulic fill following the same sequence as described under Slope Preparation (a) above. The construction of the outer face would be similar to that described under Slope Preparation (b). Above water level the construction would be in most respects exactly as for D but, if necessary, with containment being provided for the hydraulic core fill on the landward side in addition to the river side.

Hydraulic landfill between Ranigram Groyne and the existing town protection.

The timing of award has an important influence on the progress of the reclamation of Section B. In Figure 5.1 mobilization is shown as commencing in July 1994, which is a favourable scenario in that it gives the Contractor a good part of the subsequent dry season for setting up.

Assuming reasonably swift mobilisation of a dredger, reclamation of Section B should be well advanced by the start of the following monsoon season. Work on reclamation can commence even before the river level starts dropping, which may also provide an opportunity actively to train the river thalweg to suit the construction in the first season. In terms of dredger output in soft material the volume required (600,000 m^3 approx.) is small and no more than a few weeks work is required. The area is sheltered - a considerable accretion of sand has already occurred - and the practicable eastward limit will be determined only by the stability of the outward slope of the fill (the assumed 1 in 10 slope) as it extends beyond the nose of Ranigram Groyne and sand is carried away by the current. By commencing at the inland corner and working towards the river, it should be possible to advance the eastern limit of reclamation to within say 150 to 200 m of the axis of Section C (see Figure 5.4).

As the reclamation level of + 13.50 m PWD is, like the rest of Sirajganj, below normal wet season peak water levels, the Contractor will need to construct a substantial bund at or near the eastern limit of reclamation to protect the reclamation area, and the site facilities thereon, from the risk of inundation.

As the award date becomes later, so the achievable extent of reclamation prior to the onset of the monsoon season reduces. As noted earlier the cut-off date for award would appear to be about October in any year.

Most probably Section C will be built up as a bund as described above, starting from the junction with Ranigram Groyne or from Section D. It will not be necessary to gain access from Section B, although if conditions are benign this could be an alternative avoiding the need to return to finish the reclamation on completion of Section C.

Assuming, however, that it is necessary to return to Section B, then the volume remaining between the bund forming Section C and the area reclaimed initially will be relatively small. Filling would most likely be undertaken by trucking and/or dozing from a stockpile of dredged material pumped to land, the precise form of transport depending on the location of stockpile. To try to fill the enclosed area hydraulically by pumping directly from the dredger would entail significant difficulties in the disposal of waste water and silt.

5.7.3 Particular Conditions at Sariakandi and Mathurapara

Detailed descriptions of these works are given in Section 4.3. Briefly they comprise:

(a) upgrading of the existing Kalitola Groyne.

(b) construction of two new hard-points 2.5 and 5 km downstream of Kalitola Groyne.

(c) construction of low earthfill embankments, referred to as cross-bars, linking the hard-points to the main BRE embankment.

Construction Programme - Permanent Works

Because of the ever-present risk of the Brahmaputra breaking through to join the Bangali River, it has been decided that both hard-points, including their cross-bars, and the upgrading of Kalitola Groyne should be completed within one construction season. This will entail construction proceeding concurrently at a minimum of two of the three sites at any time. The preferred order of construction will depend on the pattern of channel migration and bank erosion at the time and cannot be specified now. For a June 1994 start, a 24 month contract period is envisaged.

Construction Methods

Recovery and Reuse of Concrete Blocks

The methodology of recovery and reuse of existing concrete blocks at Kalitola Groyne will be similar to that described in the previous section for Sirajganj.

Kalitola Groyne

The upgrading of the Kalitola Groyne will involve the same sequence of tasks as described for the Ranigram Groyne Nose but additional constraints will be imposed by the proximity of homesteads, restricted land access and the use of the small creek immediately to the north by fisherman and country boats providing passenger and livestock ferry services for the char farmers.

Sariakandi and Mathurapara Hard-Points

The construction of the two hard-points will follow the pattern described for standard revetment, without the added complication of recovery of existing armour material. It may prove possible largely to avoid the building up of the slope profile by the very much simpler procedure of trimming back the upper bank instead. However provision has been made for utilizing the material dredged to form the trench for the falling apron for some limited building up.

It will be left to the Contractor to decide whether he constructs the upstream termination in the dry or in the wet. The former will entail substantial dewatering effort but facilitate the laying of the geotextile filter and overlying armour. In the latter case the sequence of activities to be followed will be identical to that for the standard revetment but greater control over excavation and placing will be required to achieve the curvature. On completion of the armour layer, the depression inland of the bankline will be refilled using sand with a low silt content below water level, possibly using excess material from the dredging. Above water level more silty material may be used and the same upper soil must be replaced near the surface. The soil is to be compacted to the same density as the undisturbed soil and finished off to the same level as the original field.

Excess material from the termination excavation may be used for forming the compacted fill under the upper part of the revetment and for the cross-bar. Subject to agreement from the landowners, excess material from the dredging may be used to build up the land immediately behind the revetment, thereby facilitating drainage of this area and providing a secure flood free area of land.

5.7.4 Particular Conditions for the Phase 1B and Phase 1C Works

Each contract for the works at the remaining priority locations will include:

(a) two hard-points

(b) construction of cross-bars linking the hard-points to the main BRE embankment

Construction Programme - Permanent Works

Except potentially for the Naodabaga/Sariakandi North Contract, the critical nature of the Bangali break through scenario does not apply, and the case for completing construction in the river in a single dry season is less strong than at Sariakandi and Mathurapara. The scope of works, however, is substantially less than at Sirajganj and rather less than at Sariakandi/Mathurapara (no Kalitola Groyne). By working simultaneously on the two hard-points, an experienced contractor should be able to cope without major difficultly with the task of completing construction in the river in a single dry season. The additional costs implicit in retaining the site establishment for an additional year in order to allow two dry seasons for working in the river would seem far to outweigh any advantages. For a start in June of a particular year, therefore, a 24 month contract period is envisaged.

71

Construction Methods

Whilst each location will have its own particular problems regarding logistics, construction at each site will follow broadly similar lines, as described earlier for standard revetments, but perhaps incorporating the benefits of experience won during previous hard-point construction. Dredging, filling and profiling will follow the same procedure as outlined for Sariakandi and Mathurapara.

72

6. ENVIRONMENTAL CONSIDERATIONS

The environmental considerations relating both to the short term impacts arising from construction itself, and the longer term effects of implementing a programme of river training measures, form a separate annex to the Master Plan Report, Annex 3 - "Initial Environmental Evaluation". They are therefore not discussed at length in this chapter although sight should not be lost of their importance.

At each stage of construction, i.e. prior to the letting of each contract or each parcel of contracts, a full Environmental Impact Assessment (EIA) must be undertaken. Such assessment will review in detail the specific environmental concerns at each site as well as the cumulative effect of that particular parcel when taken in conjunction with the works previously implemented.

The EIA should also spell out the specific measures which will be taken to mitigate any adverse impacts which may otherwise result during construction or subsequently. An EIA dealing with the Priority Works at Sirajganj, Sariakandi and Mathurapara has already been issued (in July 1992). This contains an action plan for monitoring and mitigating impacts and lists the relevant contractual clauses by which the action plan will be effected. For convenience, the Action Plan is reproduced as Appendix B to Annex 3.

A most important aspect of the construction of works on the river bank is the effect on people dwelling in the vicinity, especially those who will be displaced by the works. These aspects are discussed in Annex 1 to the Master Plan Report, "Sociological Considerations", and in Annex 3. Well before construction commences at any location, a detailed Resettlement Plan will have to be prepared, describing in detail which people will be affected, how they will be effected, and the steps which will be taken to protect their interests and resettle them. A tenet of this programme is that the population displaced by a project should benefit from it.

The draft Resettlement Plans for the Priority Works were issued in December 1992.

7 3

7. ICB CONTRACT MANAGEMENT

7.1 Contract Document

Once a contractor has submitted a tender, and that tender has been accepted, a contract exists between the two parties, namely the Contractor and the Employer. The tender will have been prepared on the basis of the tender documents issued by the Employer, and the documents forming the Contract, which will usually include the greater part of the tender documents, will be those listed in the General Conditions of Contract or in the Conditions of Particular Application. These documents are generally referred to as the Contract Document, and they define the responsibilities of the parties to the Contract.

Draft tender documents for the Priority Works contracts (i.e. Phase 1A) were submitted to the BWDB in August 1992 and have recently been revised. Tender Documents for the Phase 1B and Phase 1C contracts have been prepared along similar lines, and are presented as Annex 6 of the Master Plan Report.

The river training works will be tendered by International Competitive Bidding (ICB). The conditions of contract selected for this purpose are the widely used Conditions of Contract for Works of Civil Engineering Construction published by the Federation Internationale des Ingenieurs - Conseils. The edition currently in use, and on which the tender documents are based, is the fourth edition (1987) reprinted in 1988 with editorial amendments. These conditions of contract are customarily referred to as FIDIC-IV.

7.2 The Contractor's Responsibilities

Quality Control

Under FIDIC-IV, the Contractor is responsible for the quality of his own work and for ensuring that the end product is in accordance with the Specification. The Specification (Clause 209) requires that he draw up a Quality Plan in accordance with BS 5750, implicit in which is that he will have to prepare written Procedures which will cover the sequence of operations to be followed for each key activity, the checks and controls that will be built in to ensure that tasks are completed in the correct sequence and that approvals are obtained where required before proceeding to the next task.

The minimum level of testing is either covered by the relevant Standard quoted in the Specification or detailed in the document. Construction tolerances are also set out in the Specification.

Land Acquisition for Temporary Works

The Contractor will be solely responsible for all arrangements relating to the acquisition of land for temporary works such as site accommodation for the Engineer's and his own staff, offices, workshops, casting and storage yards, wharfs, haul roads and the like. However he will have to obtain the approval of the Engineer for both the siting and the layout of such facilities and these must be such that they do not cause undue nuisance to local residents. Liaison by the Contractor with local authorities is essential in this regard.

Site Accommodation and Facilities

The Contractor will be required to submit to the Engineer detailed designs for all domestic and office accommodation for himself and the Engineer, all of which is to be supplied through the Contract. For the Engineer's accommodation he will also submit for approval details of the furnishings to be provided and also the major items of office equipment.

Provision has been made for site accommodation to be built for all professional and technical personnel on the Engineer's staff. The Contractor will be responsible for his own arrangements but the standards will have to comply with the relevant national regulations. In particular he will have to demonstrate that he has made adequate provision for all services, control of insects and safe disposal of liquid and solid waste.

Temporary Works

The design and construction of all temporary works will be the responsibility of the Contractor but he will be required to submit his detailed proposals to the Engineer for approval prior to their implementation. This will include the design of formwork and falsework, cofferdams and all facilities such as temporary wharfs. Temporary structures, such as groynes for the diversion of flow from working areas, and dewatering measures also fall into this category.

All temporary works must be dismantled on completion of the Permanent Works, unless the Employer, in this case the BWDB, specifically wishes to take over any parts and reaches specific agreement with the Contractor to this effect. The Contractor may also, with the approval of the Engineer, reach agreement with the owner of property that he has leased, or otherwise temporarily acquired, to assign any part of the temporary works to that landowner on condition that they do not represent an environmental hazard.

Borrow Areas

Certain areas designated for borrow have been shown on the Drawings and these constitute part of the Site that the Employer undertakes to provide free of charge for the sole use of the Contractor, during the duration of the contract period. The Contractor is at liberty to propose alternative sources of material that satisfy the Specification, in which case the onus is on him to carry out the necessary investigations and tests to prove to the satisfaction of the Engineer that the source is both technically suitable and, if any additional environmental impact is involved, that the mitigation measures proposed by the Contractor are appropriate and adequate.

During construction, borrow areas on the land may provide breeding grounds for insects and habitats for other diseases vectors. The Contractor is responsible for ensuring either that borrow areas are adequately drained or that suitable measures, such as the spraying of approved insecticides, are taken to control any hazard.

On completion of the works the borrow areas must be left in a condition that is suitable for beneficial agricultural or other use.

75

Timely Completion

Failure to complete the permanent works within the stipulated period may result in both direct and consequential losses for the Employer, individual land and property owners, agricultural workers and those involved in support services. Because of the hydrological character of the river, a delay of only a few days may mean that the increasing depth of water and flow velocities force the Contractor to cease work before satisfactorily finishing off the work. This may have far-reaching consequences and could lead to the occurrence of extensive damage both to the almost completed works and to the remaining unprotected area.

The Contractor is required to draw up a detailed programme at the commencement of the contract, which has to be approved by the Engineer, and to monitor his own progress against this programme. If he finds that he is falling behind the programme he must taken necessary measures, such as mobilizing additional resources, in order to recover the lost time.

If he considers that factors outside his control, and which he could not have reasonably have foreseen, have caused some delay to his work programme, he is obliged immediately to notify the Engineer to this effect and to discuss and agree on suitable measures to be taken to recover the situation. Such measures may entitle him to some additional payment, if it is demonstrated that the claim is legitimate. At all times the onus is on the Contractor to respond to such situations in a manner that is planned to minimise the impact both financially and in terms of time.

Provision is made in the Contract for liquidated damages to be imposed on the Contractor in the event that he fails to substantially complete the works in the specified time. Such sums are related to the potential direct and consequential costs to which the Employer may be exposed as a consequence of the delayed completion and are typically expressed in terms of a sum per day of delay. In this case, even a short delay may result in postponement of completion by six months and exposure to high risk during this period. The form and amount of the liquidated damages should reflect this.

Maintenance of the Works during Construction

Under earlier versions of the FIDIC Conditions the Contractor's responsibility for maintenance during the period of construction was ambiguous and led to many arguments. The FIDIC-IV terms and conditions, are much clearer in this respect: the Contractor is required to execute and complete the Works and to remedy any defects. There is thus no obligation on him to carry out any routine maintenance, other than that necessary for assuring the safety of the works in accordance with the Specification, provided that he hands the works over in good condition upon completion. Following the handing over of the Works, or any part thereof, the responsibility for maintenance lies unequivocally with the Employer, although the Contractor remains responsible for the remedy of any defect.

An important consideration in this respect is the responsibility for the care and maintenance of the works during an intervening monsoon season in the case where construction will span over two dry seasons. Specific provision has been made in order that the Contractor shall be responsible for finishing off the uncompleted works in such a manner that the risk of significant damage occurring to the part of the works that has been completed is acceptably low. This will normally involve the construction of some temporary works that will be demolished or incorporated in the permanent works when construction is resumed. The Contractor will also be responsible for caring for both the partly completed works and the temporary works and will have to satisfy the Engineer both that the temporary works are adequate and that he will have the resources available during the monsoon season to undertake emergency remedial work if the necessity arises. Among the Phase 1 works, these considerations apply only at Sirajganj.

Defects Liability Period

During the stipulated Defects Liability Period, in this case 12 months, which commences from the day of the taking over of the works by the Employer, the Contractor is responsible for making good any defects arising from poor workmanship or materials, but not for defects that are a attributable to fair wear and tear or are a consequence of the design and the environment in which the works are constructed.

In this case it is likely to be difficult to differentiate between the two. For example, the failure of a falling apron to protect the toe of the revetment from scour could possibly be due to either the development of exceptional scour or to the incorrect distribution of the armour material as placed in the apron.

After the event, it will probably not be feasible to determine which of these was the cause. In such situations, if the Contractor is still on site with the appropriate plant, the options are either to instruct the Contractor to carry out remedial work, for which he will request payment, or for the BWDB maintenance unit to undertake the work, with support from the Contractor through the provision for training.

Training of BWDB staff

Provision is made in the Contract for the Contractor to provide specific formal training to personnel seconded by the BWDB for this purpose. The training will be primarily on-the-job in nature and the objective will be to familiarize the BWDB staff with the use of the type of plant and equipment that will be appropriate for maintenance of the works and which will be procured for this purpose as a part of the Project.

7.3 The Employer's Responsibilities

Possession of Site

The Employer will be responsible for providing the Contractor with free possession of the Site on the date or dates shown in the Contract. This means that the Employer must himself have legal possession of the Site at the time and be in a position to assure the Contractor of unimpeded access to and movement within the Site for the purposes of constructing the Works. If there are any limitations on this possession they have to be clearly spelt out in the Contract.

The Contractor must under the conditions of the contract be provided with legal access to the place of work from a suitable public thoroughfare; for practical purposes this public thoroughfare should be adequate for the transport of the type of plant that the Contractor will require to undertake the work. In this case the nature of the work is such that access by river

77

is the most practicable and the BWDB's responsibility is to ensure that the statutory standards of navigation are available to the Contractor and that the issue of any permits is facilitated.

In this case the presence of squatters on the existing BRE in the vicinity of Sirajganj is such a special condition. As a part of the Contract the Contractor is required to resettle the squatters and must have possession of the Site in order to undertake this task. However, it is not his responsibility to enter into any form of consultation with the squatters with regard to the timing of the resettlement; unless this is stipulated in the contract he must be at liberty to schedule the resettlement to suit his work programme. The responsibility for prior consultation and liaison with the other authorities lies with the Employer; following the award of the Contract, the Resident Engineer will act as the line of communication between the Employer, the Contractor and the concerned authorities. Although the Contractor will be encouraged to establish his own informal direct line of communication with the squatters to ease the resettlement process, the formal communication with the squatters will remain the responsibility of the BWDB. Similar considerations apply to other sites where the resettlement of people, landowners or squatters, is involved, and plans for their resettlement must be drawn-up. Draft Resettlement Plans for the Priority Works sites have already been issued (December 1992).

Failure by the Employer to give the Contractor unimpeded possession of the Site, including access by the routes described in the Contract, on or before the stated date(s) will provide the Contractor with a legitimate basis for a claim for an extension of time for completion of the works. Apart from possible financial ramifications, this could have far-reaching consequences with regard to the actual completion date in view of the well-defined construction season.

Visas, Work Permits and Duty Free Importation of Plant

Under the contract conditions, the Contractor is responsible for obtaining visas and work permits for his expatriate staff but the Employer undertakes to actively facilitate the process. It is important that the BWDB should establish procedures to this effect with the concerned authorities prior to the award of the Contract. It is preferable that these procedures, and any restrictions that may apply, be made known to the tenderers at the time of the pre-bid conference.

Under the terms of the Contract the Contractor will be permitted to import plant required for the execution of the Works provided that he undertakes to re-export it, or pay the duty, upon completion of the works. Although not a specifically incorporated in the World Bank approved ICB Sample Bidding Documents, the Contractor will almost certainly be required to take out a bond to this effect. The BWDB will be expected to endorse the Contractor's application, provided that the Engineer confirms that the application is reasonable, and to provide whatever assistance towards the expeditious issue of the import permit that may be within their power.

Provision of Personnel for Training

The proposed arrangement whereby BWDB staff will be assigned to receive on-the-job training with the Contractor has been outlined above. It is further proposed that BWDB staff also be seconded to the Engineer's staff in order to develop experience in the management of this type and scale of contract.

BWDB have agreed in principle to the payment of special allowances to the seconded staff. It is important that this be followed up in order that at the appropriate time funds are available, authorization has been issued and suitably qualified, experienced and motivated staff have been freed from other duties.

7.4 Land Acquisition

Contractor's Temporary Works

As described in Section 6.2, the Contractor will be responsible for arranging temporary acquisition of land for all his temporary works, including his and the Engineer's staff housing. This arrangement derives benefit from the Contractor's greater flexibility with regard to the negotiation of inducement and compensation payments than is open to the BWDB.

Acquisition for Permanent Works

The land acquisition obligations lying with the BWDB have been outlined in Section 6.3. The minimum practicable extent of the Site comprises the permanent works themselves, associated borrow areas, limits of excavation and sufficient space around them for the Contractor to manoeuvre and operate his plant and to locate site offices and access corridors. These limits are shown on the Tender Drawings so that the tenderer can plan and price his work accordingly. No land access corridor is being provided for the Contractor at the Sariakandi/Mathurapara site because the Site is directly accessible by way of the river and he is being encouraged to use this as his main route for all purposes. At Sirajganj limited road access to both north and south limits of the Site do exist and will be available to the Contractor subject to load restrictions; again, the Contractor is encouraged to make maximum use of the river for access. Particular constraints on access will apply at the other Phase 1 sites.

The requirements for permanent acquisition are limited to the area occupied by the Permanent Works themselves and a small peripheral strip for maintenance access. The remainder of the area to be designated as the Site may be acquired for the duration of the Contract only and thereafter revert to the owner.

The approximate areas to be acquired by BWDB for the Permanent Works, the areas to be temporarily acquired by BWDB to provide working space for construction (together called the "Working Area" on the Drawings) and borrow pits, and the areas likely to be temporarily acquired by the Contractor are given below for a typical hard-point. The figures relate to areas landward of the present dry season bankline.

Permanent Works	10 ha
Working Area (including Permanent Works)	17 ha
Borrow area (provisional)	8 ha
Temporary acquisition by Contractor	20 ha

At Sirajganj, the land along the existing revetment, required for construction of the revetment which will replace it, is understood to be already in public ownership and acquisition should not therefore pose a problem. Although the 25 ha of fill to be placed in Section B can be regarded as Permanent Works, the land, on completion, may revert to its original owners. This area can therefore be regarded as temporary acquisition by BWDB. If BWDB subsequently wish to acquire part of it for a permanent base for their Survey, Evaluation and Maintenance Unit, that is a separate issue to be dealt with nearer the time.

7.5 Taking-over of Contractor's Facilities by the Employer

There is no provision made in the Contract for the Employer to take over any of the Contractor's plant, equipment, transport or accommodation. All imported plant, vehicles and equipment must therefore be re-exported on completion of the works and all temporary works dismantled and the site reinstated.

If the BWDB decides that it would like to take over any part of the plant, equipment, vehicles or temporary works, then this will be a matter for negotiation directly between the BWDB and the Contractor.

7.6 Selection of Armour Material

The alternatives open for use as armour material and their respective merits were outlined in Chapter 3. Since the dependency on imported rock would place a high premium on the continuation of an open border and also provide suppliers with an undesirable strong base for price negotiation, it has been agreed that the tenderers be required to price alternative Bills based on quarried rock, brick aggregate concrete blocks and stone aggregate concrete blocks respectively. This arrangement also provides the tenderers with the flexibility to offer any particular comparative advantage that they may have with respect to their access to specialized plant or markets. Finally, although one of the three will be selected at the time of tender evaluation, the other two alternatives will offer a useful basis for a variation should circumstances change and the first selection prove untenable.

7.7 Materials Left to the Tenderers' Discretion

All materials are specified in terms of compliance with specified performance criteria and material properties. Within these limits the Contractor is free to submit details of any materials for the approval of the Engineer.

The only materials left entirely to the tenderers' discretion are those that will be utilized in the temporary works.

7.8 Quality Assurance

The Contractor will not be required to be registered in accordance with the provisions of BS 5750 but he will be required to comply with the provisions, particularly with regard to the preparation of a Quality Plan and written Procedures. This is important in the context of a project in which a high level of quality control is required in conditions that make quality monitoring difficult while the intensity of work will be unusually high during the limited construction season.

The Engineer should similarly comply with BS 5750 and prepare and follow a Quality Plan. Personnel seconded from BWDB will be expected to comply in this respect and part of their training will be in the application of QA to construction supervision.

80

The Engineer's role in the exercise of quality control on Site is summarized in Appendix J.

7.9 The Engineer's Responsibilities

The responsibilities of the Engineer are commonly divided into those that constitute direct support to the Employer with respect to both technical and contractual issues and those that are more concerned with the day to day supervision of the Contractor's activities on the Site. In practice the distinction lies in those powers vested in the Engineer that are delegated to the Engineer's Representative, commonly but not necessarily synonymous with the title Resident Engineer, and those that are retained by the Engineer. Where a separate office is established for the purposes of supervision-in-chief, as is proposed here, and more than one construction contract is involved, the Engineer will normally delegate to the Chief Resident Engineer certain of the powers not delegated to the Resident Engineer.

Supervision-in-Chief

The activities that fall within this category will be:

- Specifying and supervising surveys and investigations and analysing the results during the period between the award of the Contract and the start of the main construction season; these will be required to provide both the baseline data for subsequent impact monitoring during construction and the current morphological information required for the finalization of the layout of the Works.

- Preparing updated designs to suit the actual reach morphology and bed bathymetry at the end of the monsoon season immediately preceding the start of the main construction period.

- Responding to queries from the Contractor on design and material specification issues.

- Preparing Variation Orders for unforeseen work not covered by the BOQ that may arise during the construction period, and Provisional Sum Orders for work not possible to detail prior to tender.

- Foreseeing potential difficulties and bottlenecks and advising the Employer on appropriate action to minimise the probability of situations arising that could lead to the submission of a claim by the Contractor.

- Advising the Employer on contractual issues, including the management of claims and any requests for extension of time submitted by the Contractor.

- Carrying out Quality Assurance audits.

- Certification

81

On Site Supervision

- Monitoring the Contractor's compliance with the Contract terms and conditions.
- Monitoring the Contractor's compliance with the Specification, including provisions for environmental impact mitigation during construction.
- Monitoring the Contractor's compliance with his Quality Plan
- Monitoring the quality of materials and compliance with construction tolerances, including the supervision of surveys and tests for quality audit purposes.
- Measurement of the Works and checking of the Contractor's submissions for interim payments.
- Progress monitoring, forecasting of bottlenecks and exploring with the Contractor means of addressing these within the provisions of the Contract.
- Checking of the Contractor's setting out, preparation of foundations, fixing of reinforcement steel and the like.
- Checking of the Contractor's method statements and proposals for temporary works and referral of these to the Engineer for approval; monitoring the Contractor's compliance with these statements.
- Preparation of record drawings.
- Responding to emergency situations as they arise.

7.10 Contract Supervision Organization

The intensive pace of construction that is implicit in the quantum of work that must be a completed in the available construction windows will mean that supervision of the Works will be very a demanding task, calling for substantial experience of the supervision of large international contracts of this type and the ability to respond rapidly to unforeseeable difficulties as they arise. This level of experience and expertise is normally only to be found within a firm of international consulting engineers and there are clearly major advantages in engaging for this purpose the same firm that prepared the engineering designs and is thoroughly familiar with the conditions.

The Engineer will then be a nominated individual, typically a Director or equivalent of the firm, who has the requisite experience and can readily call upon the technical and contractual specialist advice available within the firm. The Engineer will formally delegate those majority of his duties and responsibilities to one or more suitably qualified representatives resident in Bangladesh but will retain certain key powers and responsibilities.

It is recommended that for these contracts the organizational structure for the Engineer's Representative and his support staff be as shown in Figure 7.1, which is based on the Priority Works contracts. Each of the Contracts will require a suitably experienced engineer, who will

8V

be delegated those of the Engineer's powers and responsibilities that are necessary for the day-to-day running of the work on site. This post is commonly referred to as the Resident Engineer (RE) and he will be responsible for the tasks described under Site Supervision in the previous Section, while the Chief Resident Engineer (CRE) will be responsible for the execution of the tasks described under Supervision-in-Chief, other than those where the responsibility is retained by the Engineer. Normally the CRE will be a senior member of the firm and the Engineer will delegate to him the majority of powers not delegated to the RE, but retaining powers such as those under Clause 2 (Engineer and Engineer's Representative) and Clause 67 (Settlement of Disputes).

The Chief Resident Engineer should be based in Dhaka in order to facilitate communication with the Engineer and the Employer and to have ready access to back-up services in his home office. He will be supported by a Design Engineer, in charge of a small design unit comprising engineers and draughtsmen, and a senior Contracts Engineer, who will advise on the management of Contractor's claims and other matters relating to contract law that may arise. The Design Engineer will be responsible for revising the detailed design drawings to reflect the actual morphological conditions immediately prior to the start of construction, checking the Contractor's proposals for temporary works and preparing additional working drawings as the need arises. He will thus require a very thorough understanding of the design of protection works and be familiar with the hydraulics of mobile bed rivers. This team will require the normal office accommodation and range of support services.

Each Resident Engineer will require the support of a Deputy Resident Engineer, who should be a Bangladeshi national provided that a suitable candidate is available, a Senior Construction Engineer and a Quality Assurance Manager. The Deputy RE's duties will be primarily concerned with measurement, maintaining liaison with government and other bodies, environmental impact monitoring and enforcement, the checking of the Contractor's proposals for temporary works, the overseeing of the materials testing and progress monitoring. The Senior Construction Engineer will, like the RE, be well experienced in works of the type to be constructed, and will play an important part in supervising the Contractor in the field and training and supporting the site engineers. The Quality Assurance Manager's duties will centre on the monitoring of all aspects of quality control and in particular the Contractor's compliance with his Quality Plan. Appropriate experience in roles of equivalent responsibility are the most important qualifications for the QA Manager and it is not necessary that he have an engineering degree or professional qualifications provided that his background and proven performance matches the requirements of the job.

The total staff establishment required for fulfilling the duties detailed in the previous section and the respective durations of the individual inputs are shown in Figures 7.2, Supervision-in-Chief, 7.3, Sirajganj Contract and 7.4, Sariakandi and Mathurapara Contract. Counterpart staff provided by the BWDB would be able to fill local engineer positions, depending on availability and experience. The staff organisation for the remaining Phase 1 contracts would be similar to that for Sariakandi and Mathurapara.

83

8. OPERATION AND MAINTENANCE

The provisions for operation and maintenance form the subject of a separate annex (Annex 5) to the Master Plan Report, and consequently are not discussed at length here.

The operation and maintenance of works are an important consideration during the design process, and frequently a balance has to be struck between the initial cost of a structure and future maintenance costs. The design of the Phases 1 works is based on a 1 in 100 year event and a relatively low level of expenditure on maintenance is expected.

Maintenance should be properly planned, which will require regular monitoring of the completed structures, and the availability of materials, personnel and equipment to deal with situations as they arise. A further requirement is the monitoring of river planform, particularly in the vicinity of structures, so that any potential impact on the security or effectiveness of the structures is readily detected.

It is proposed that a well equipped monitoring and maintenance unit be set up within the BWDB to:

- undertake monitoring as outlined above
- maintain adequate stocks of materials including geotextiles and armouring
- maintain the river training structures, using land based and marine plant as necessary
- undertaking additional bank stabilization measures necessitated by unexpected morphological changes or rapid bank erosion.

Detailed proposals have already been put forward for the Priority Works, in the report "Provisions for Operation and Maintenance" (July 1992). It is considered that for those works, a single unit based at Sirajganj would adequately be able to cover the three locations concerned. In the expectation of construction of more river training structures in the future, however, a Head Office/District Office structure has been proposed. With the implementation of further Phase 1 works and, looking further ahead, the long term works, more district offices will be required. The organizational aspects are considered further in Annex 5.

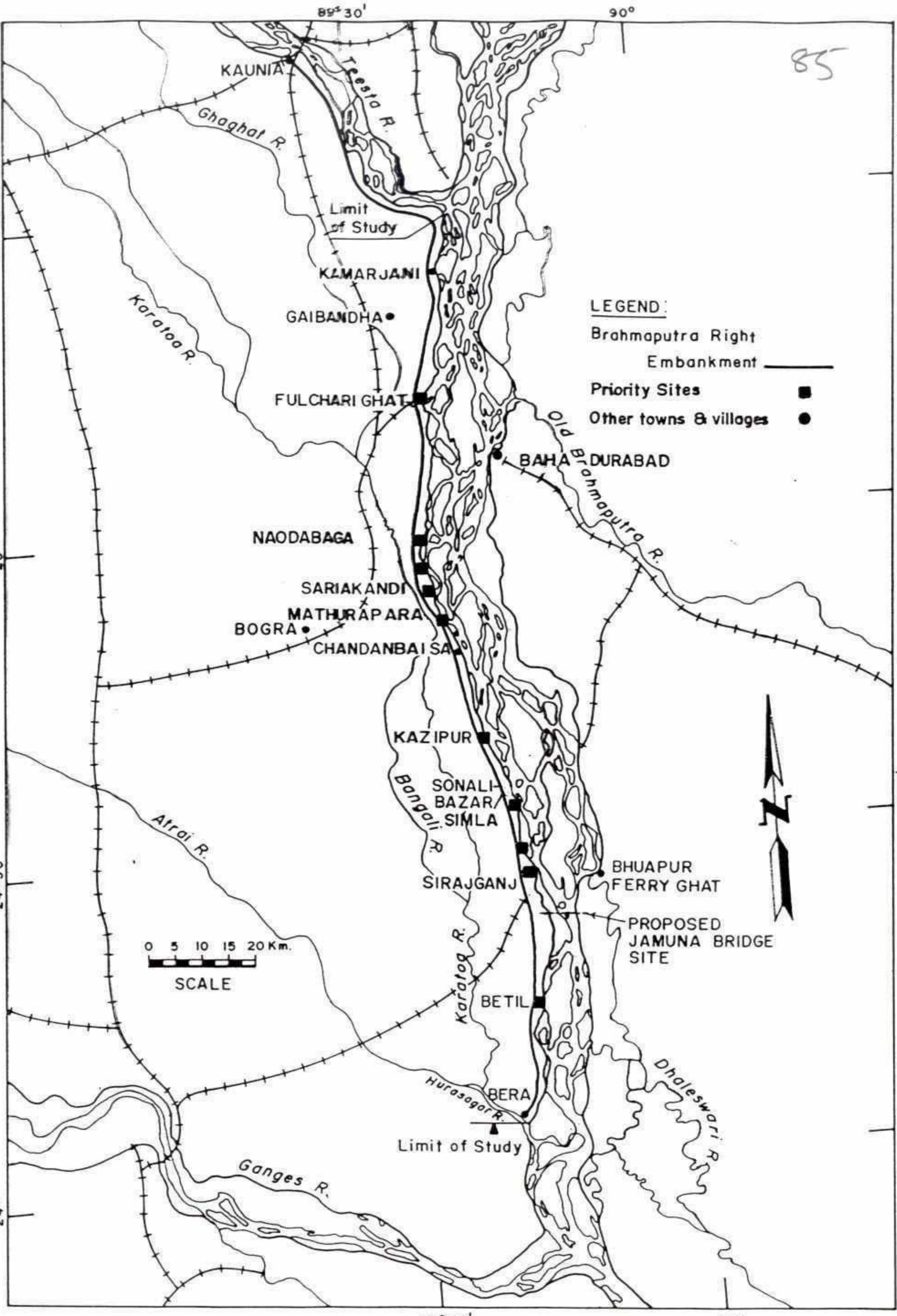
Training of the BWDB staff who will be involved in operation and maintenance forms an important part of the overall strategy. The construction contracts include provision for training BWDB personnel in construction practices, and BWDB engineers will be seconded to the consultant's team for the duration of the contract. Further guidance will be given by assigning selected contractor's and consultant's staff to the Monitoring and Maintenance Unit for a period after completion of the Works.

Alternative suggestions of extending the construction contract so that the Contractor remains responsible for routine maintenance for a period of two or more years after completion, and of letting a separate maintenance contract, are discussed in Annex 5.

89

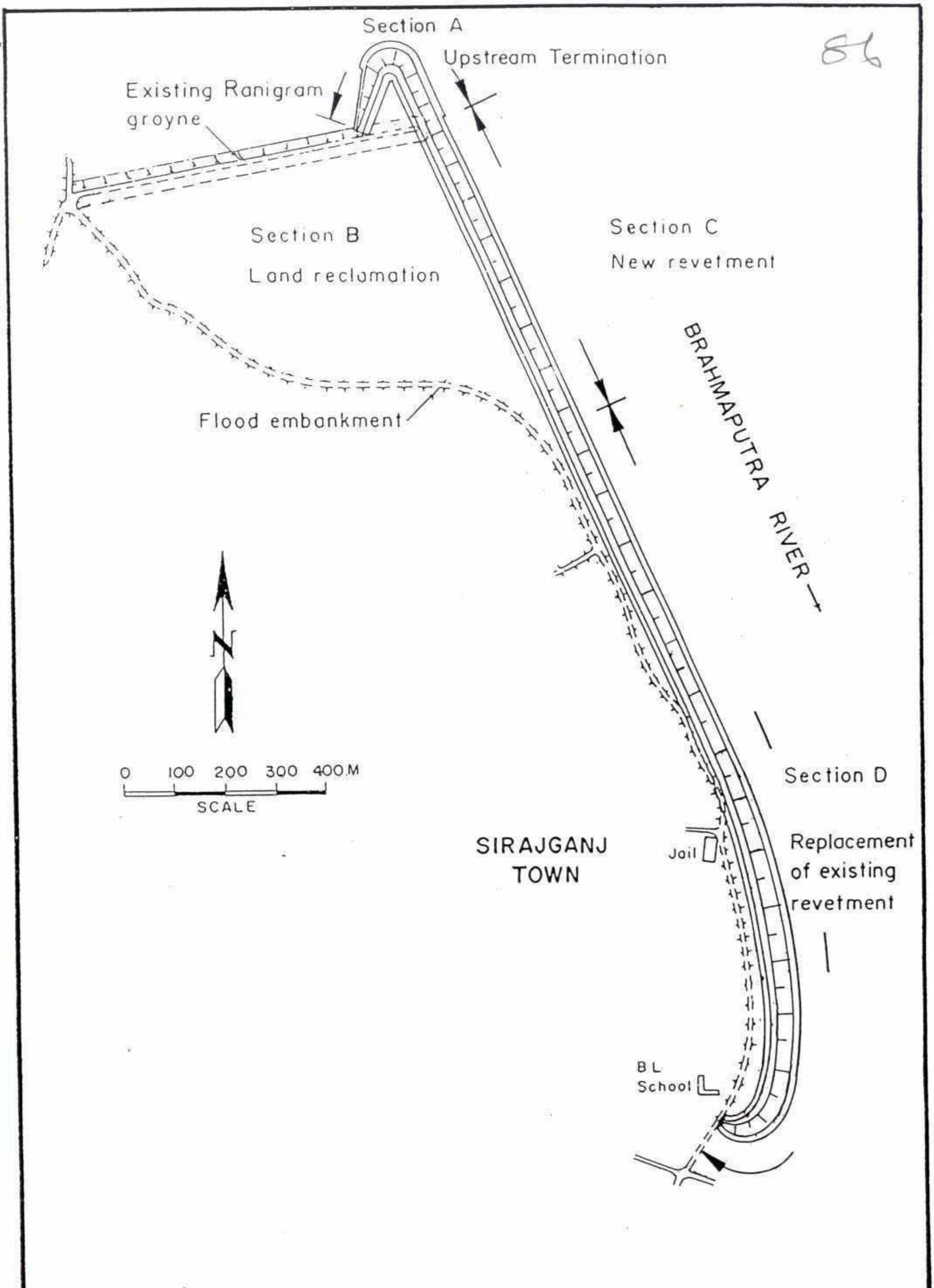
FIGURES

85



Location of Priority Sites

		Annex 4
Halcrow/DHI/EPC/DIG	*May 1993*	Figure 2.1

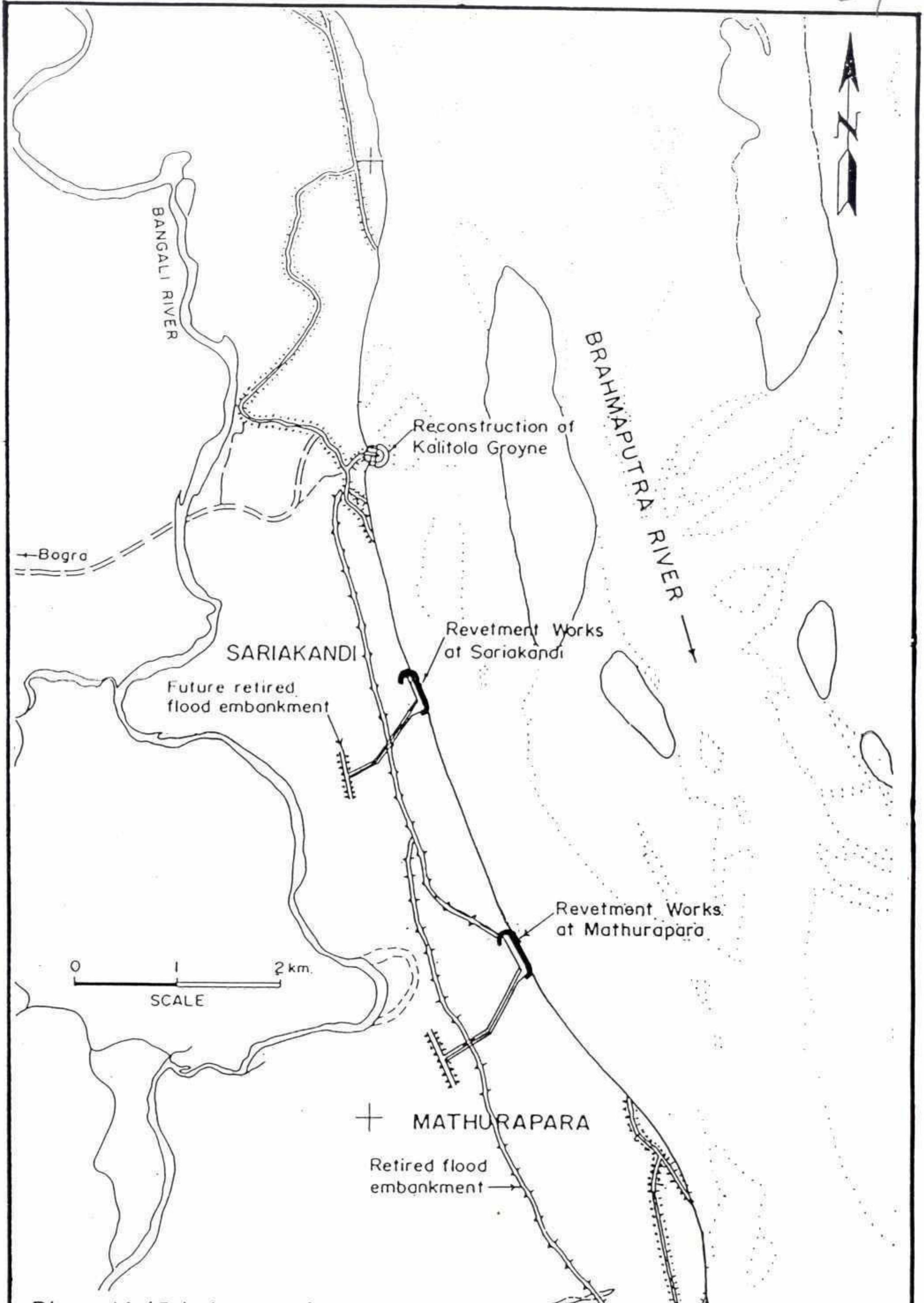


Phase 1A (Priority works)

Layout of Works at Sirajganj

	Annex 4
Halcrow/DHI/EPC/DIG May 1993	Figure 2.2

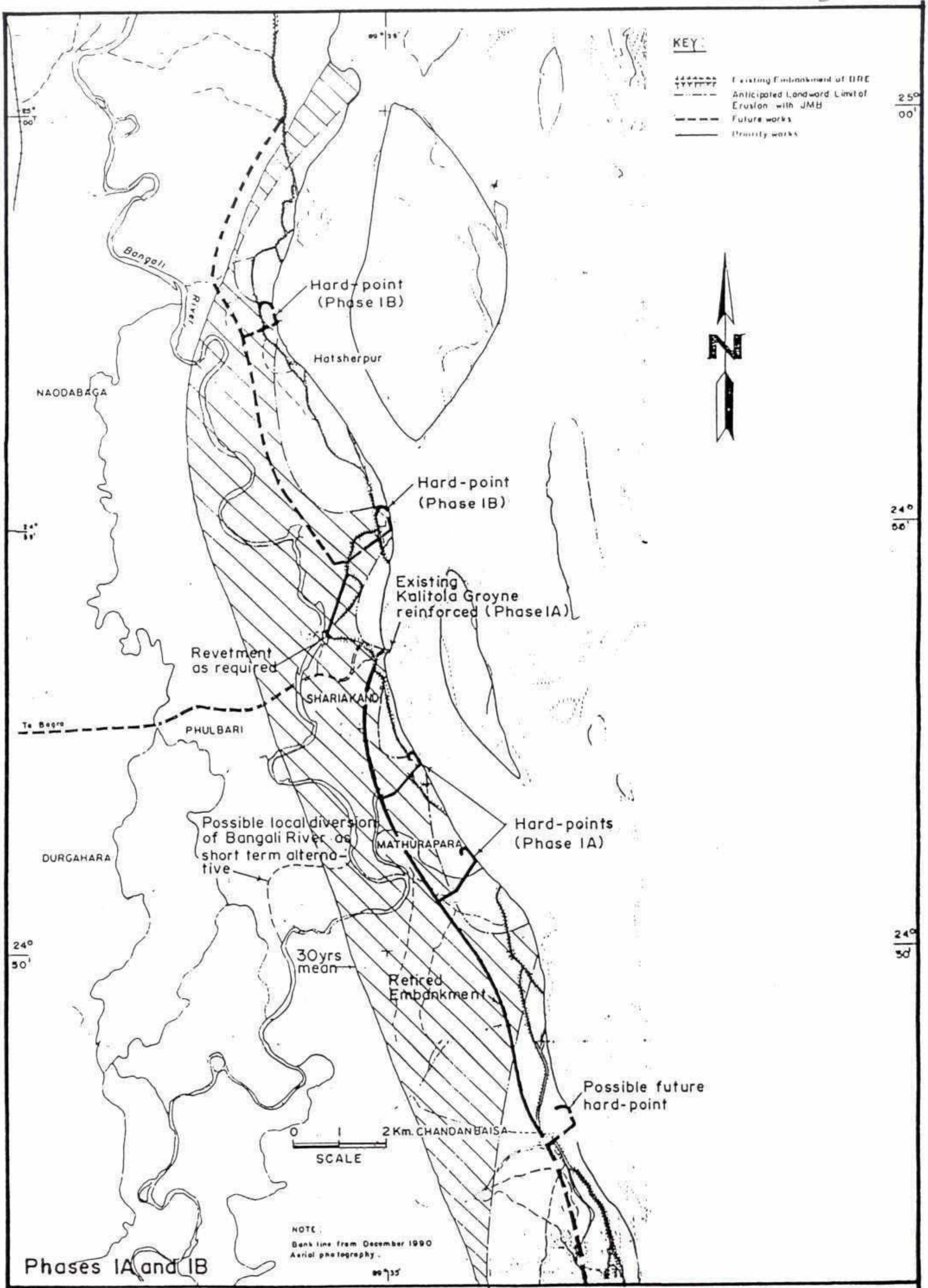
87



Layout of Works at Sariakandi and Mathurapara

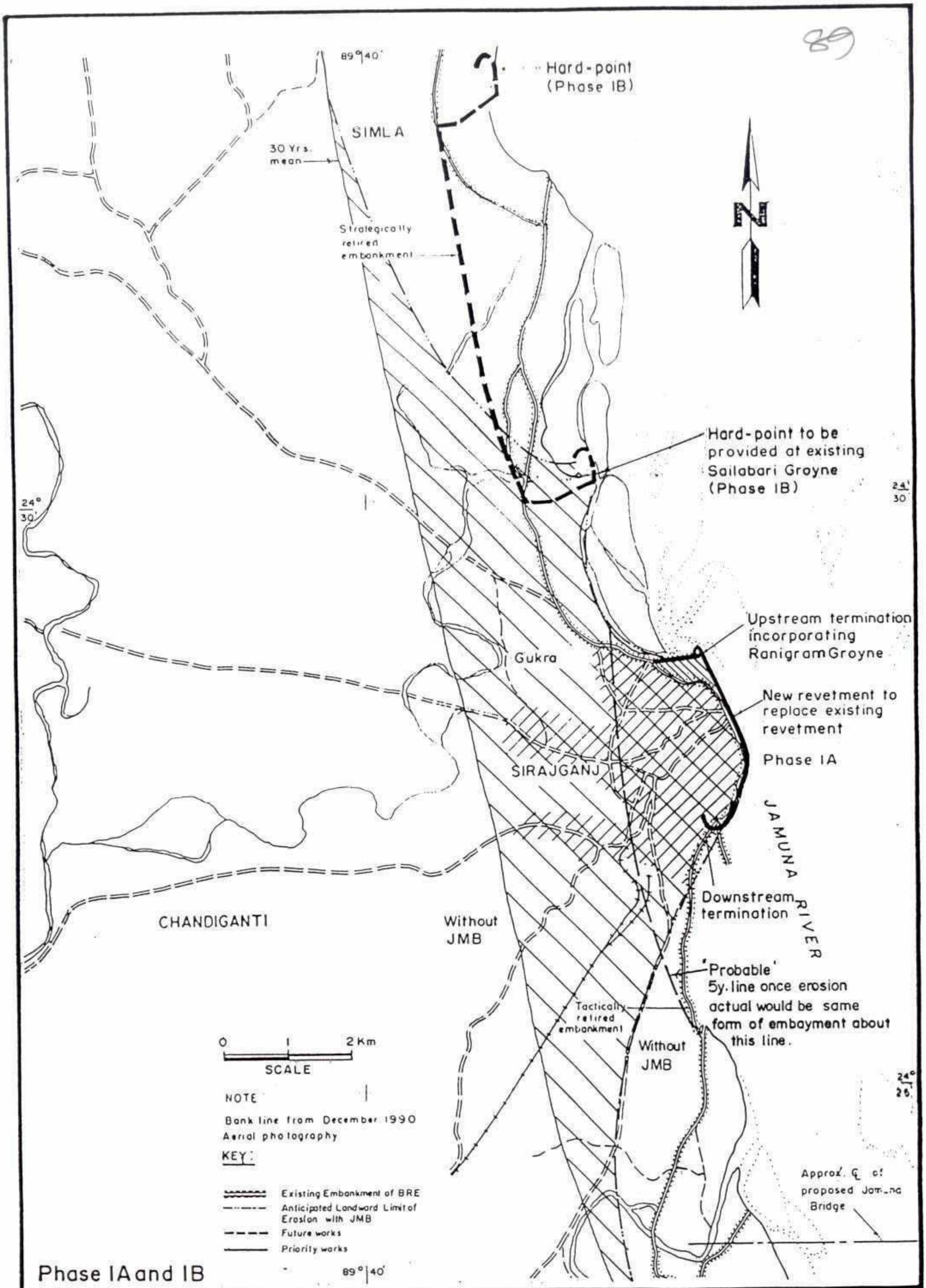
	Annex 4
Halcrow/DHI/EPC/DIG *May 1993*	Figure 2.3

88



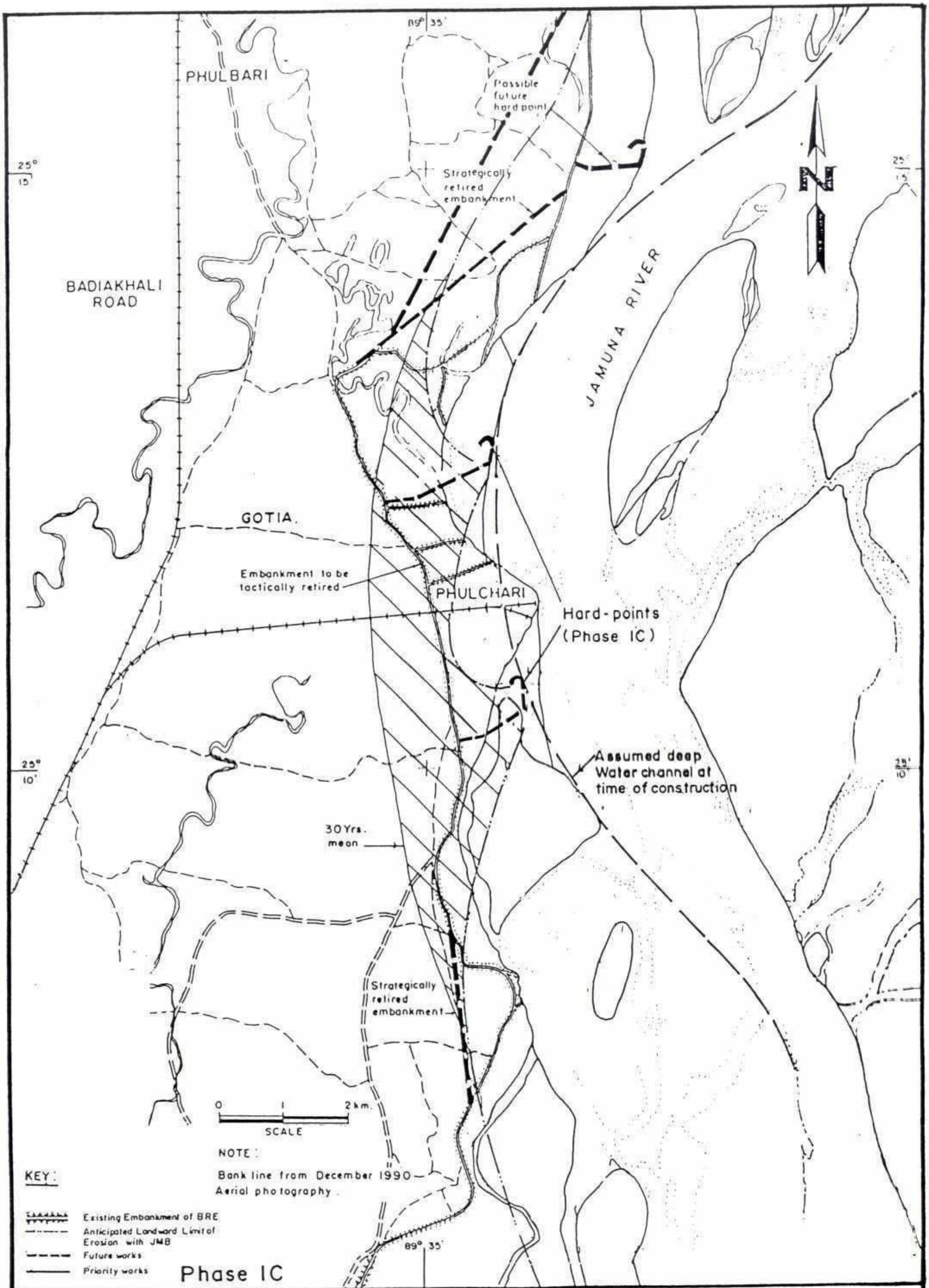
Priority and Future Works at Sariakandi and Mathurapara

	Annex 4
Halcrow/DHI/EPC/DIG *May 1993*	Figure 2.4



Priority and Future Works at Sirajganj

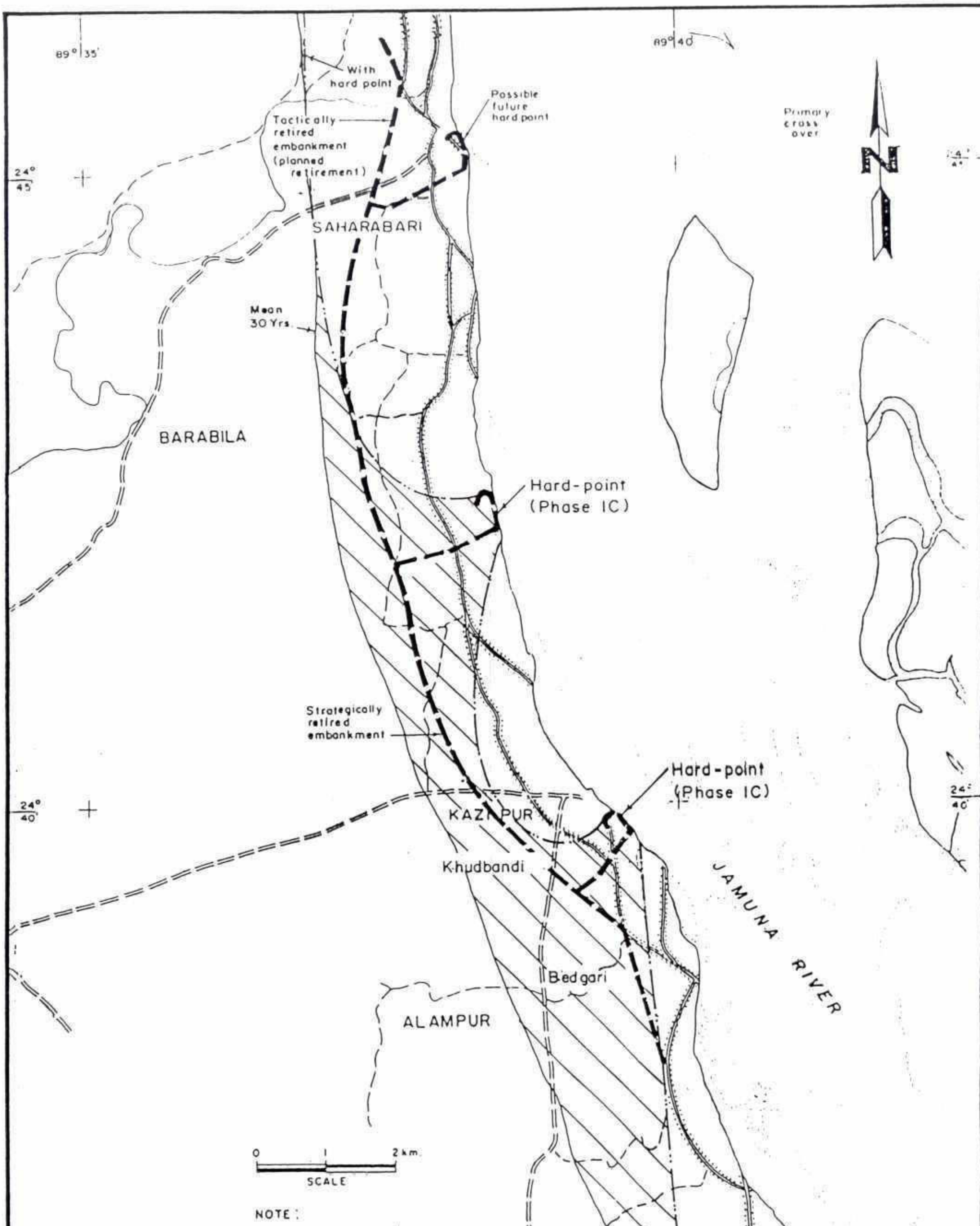
90



Priority and Future Works at Fulchari

Halcrow/DHI/EPC/DIG	May 1993	Annex 4 Figure 2.6

91



Priority and Future Works at Kazipur

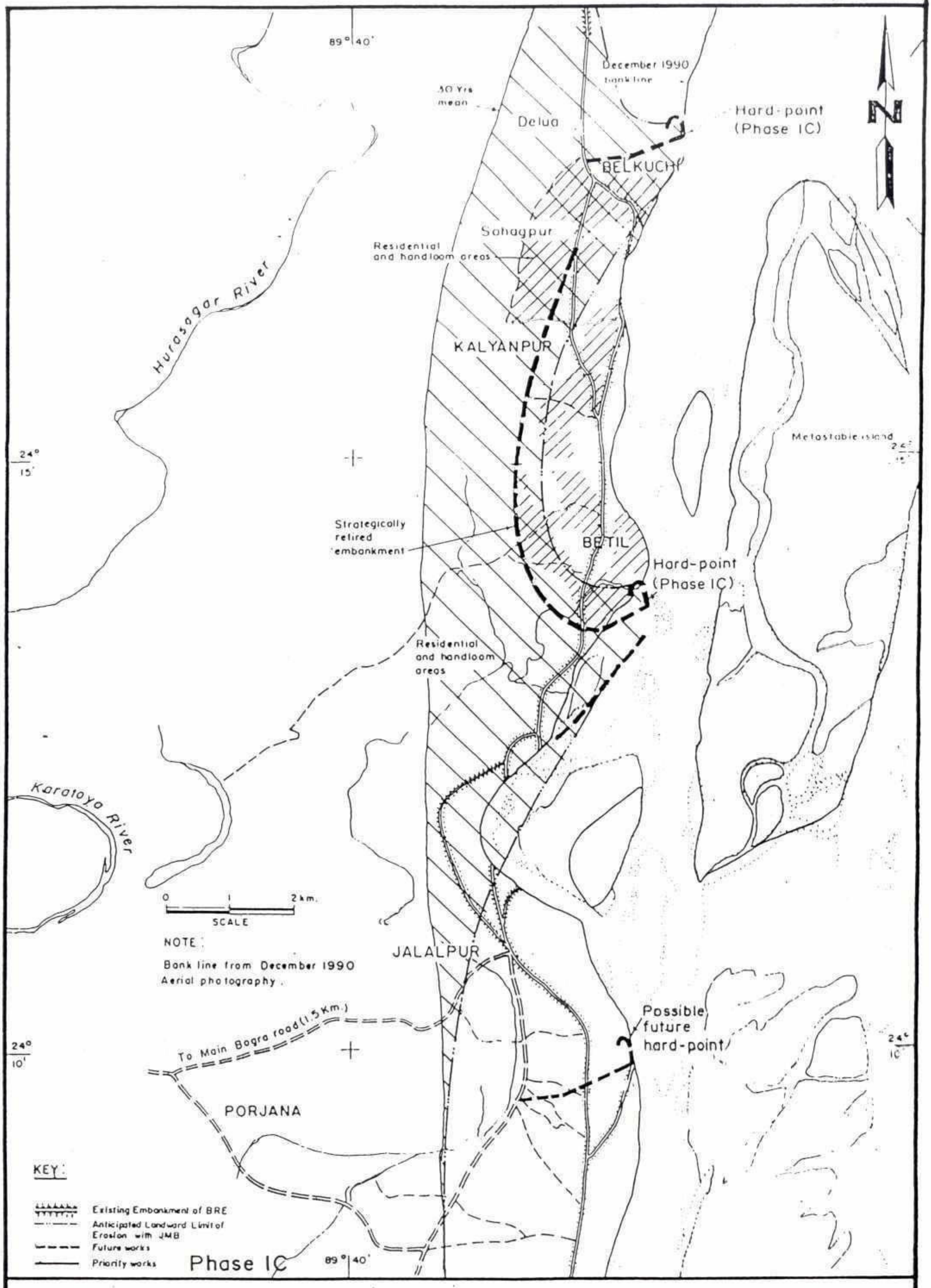
Annex 4

Figure 2.7

Halcrow/DHI/EPC/DIG

May 1993

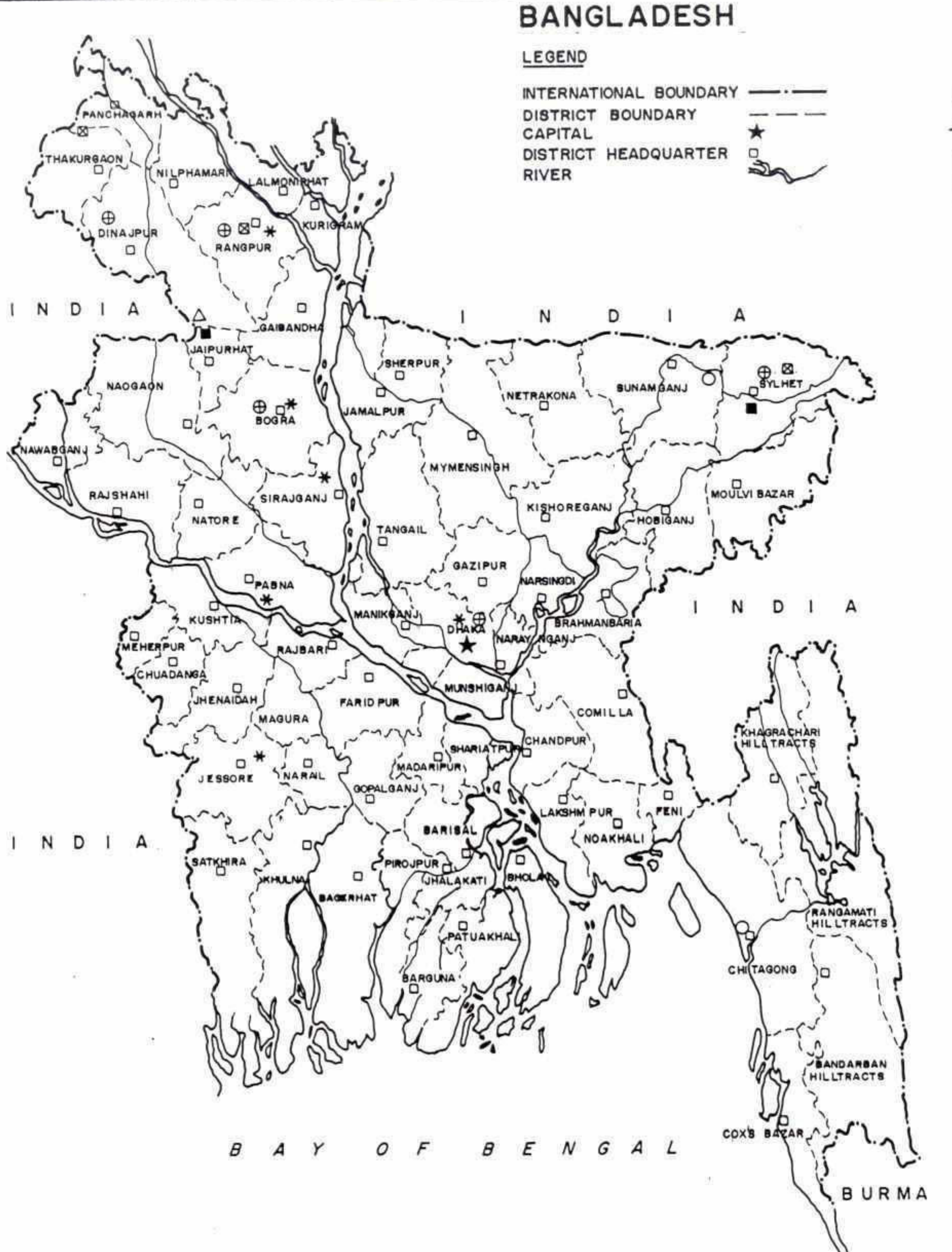
92



Priority and Future Works at Betil

Halcrow/DHI/EPC/DIG	May 1993	Annex 4 Figure 2.8

93

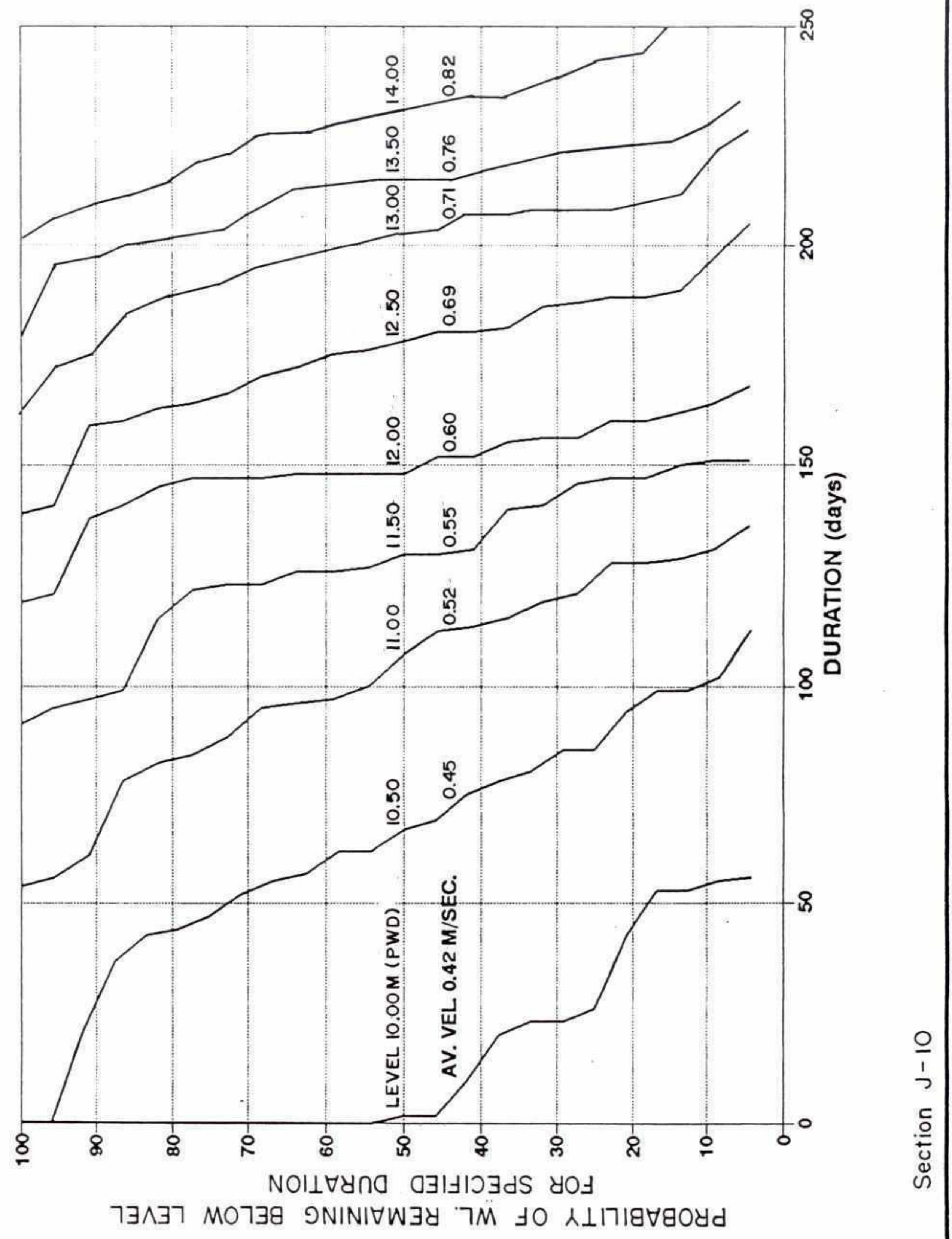


MATERIALS SOURCES

CEMENT	○
SAND	⊕
BRICK	*
SHINGLES	⊠
CRUSHED ROCK	■
QUARRIED ROCK	△

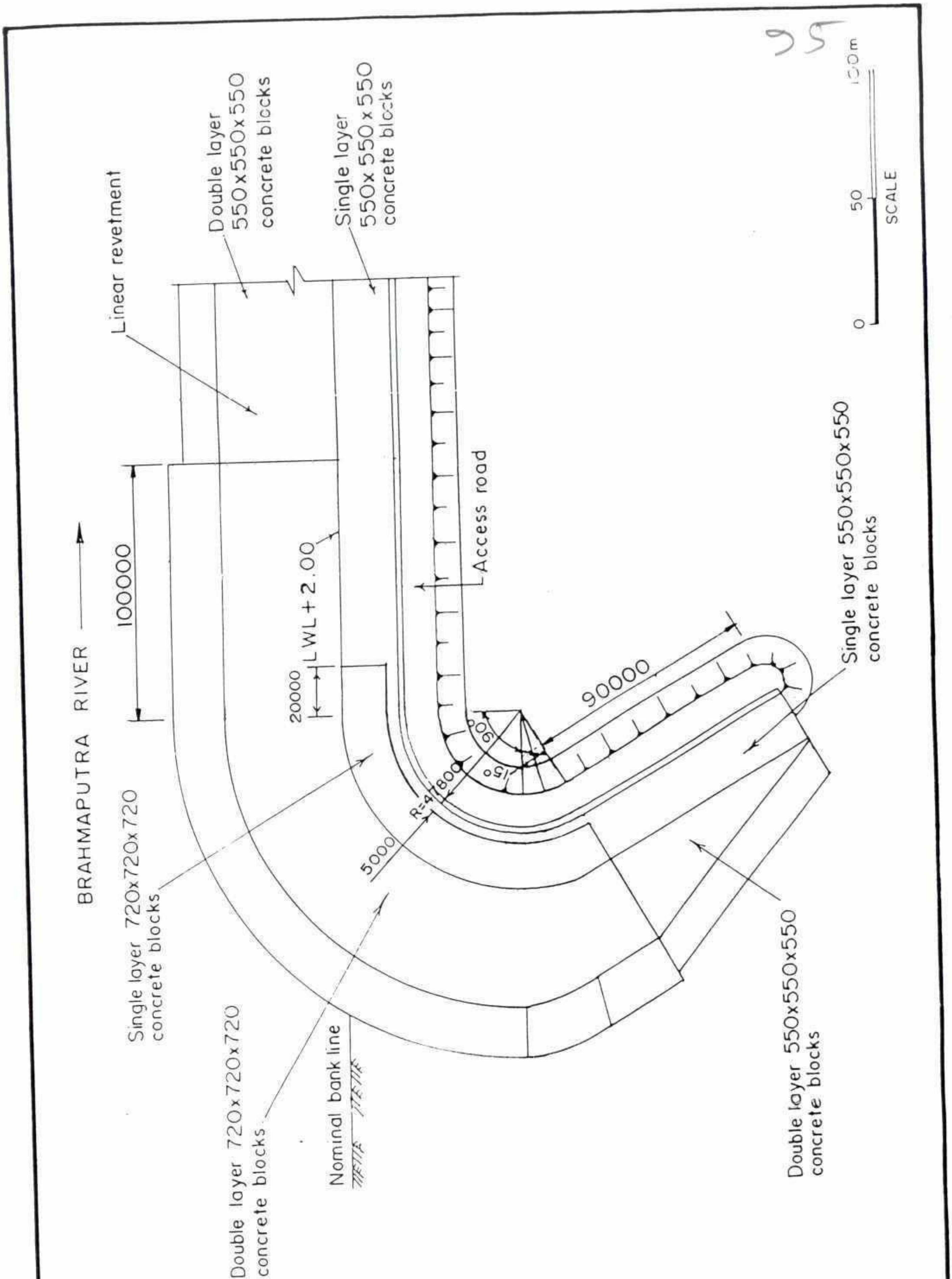
Sources of Materials

	Annex 4
Halcrow/DHI/EPC/DIG *May 1993*	Figure 3.1



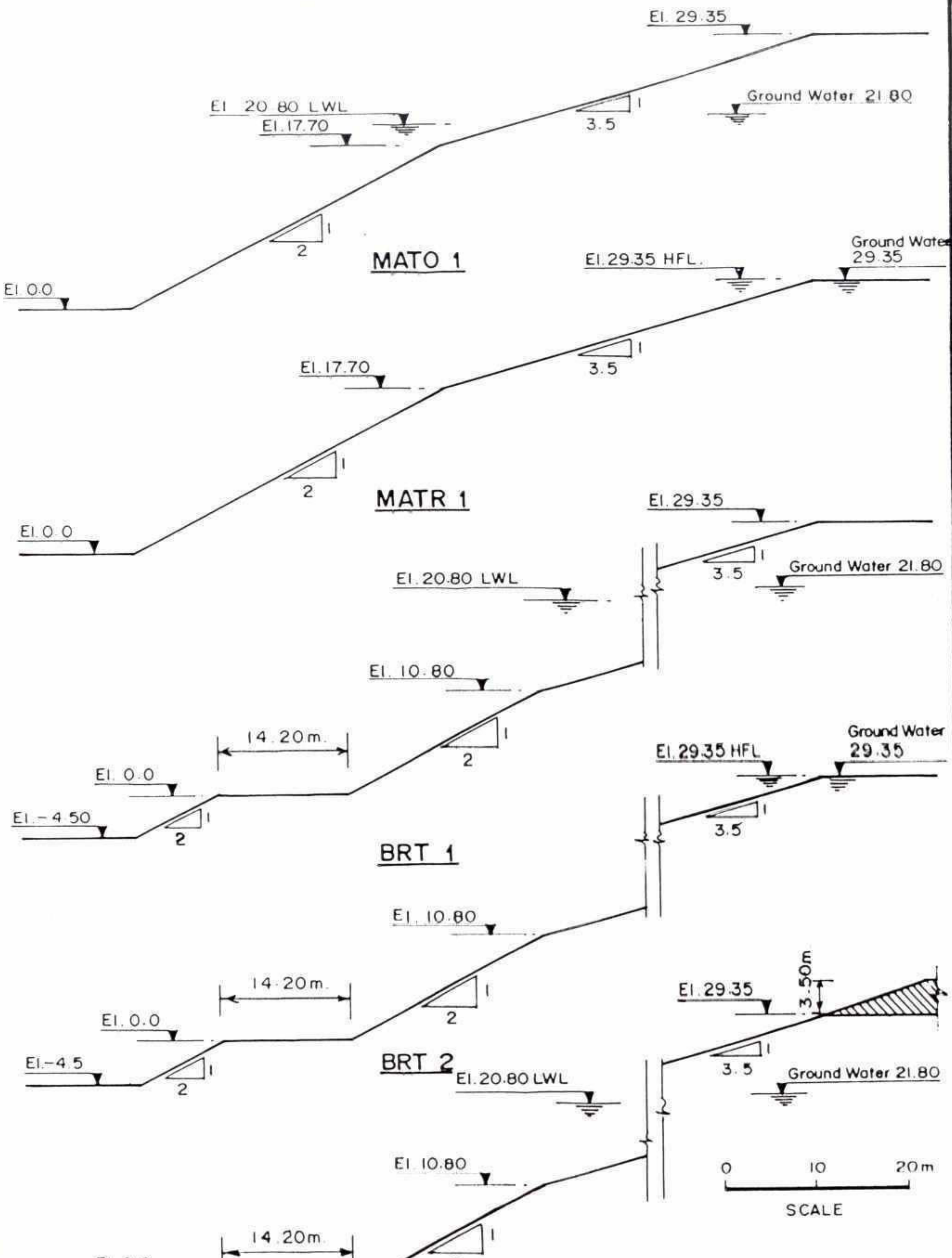
Probability of a Construction Window

	Annex 4
Halcrow/DHI/EPC/DIG *May 1993*	Figure 3.2



Plan of Upstream Termination

	Annex 4
Halcrow/DHI/EPC/DIG May 1993	Figure 3.3

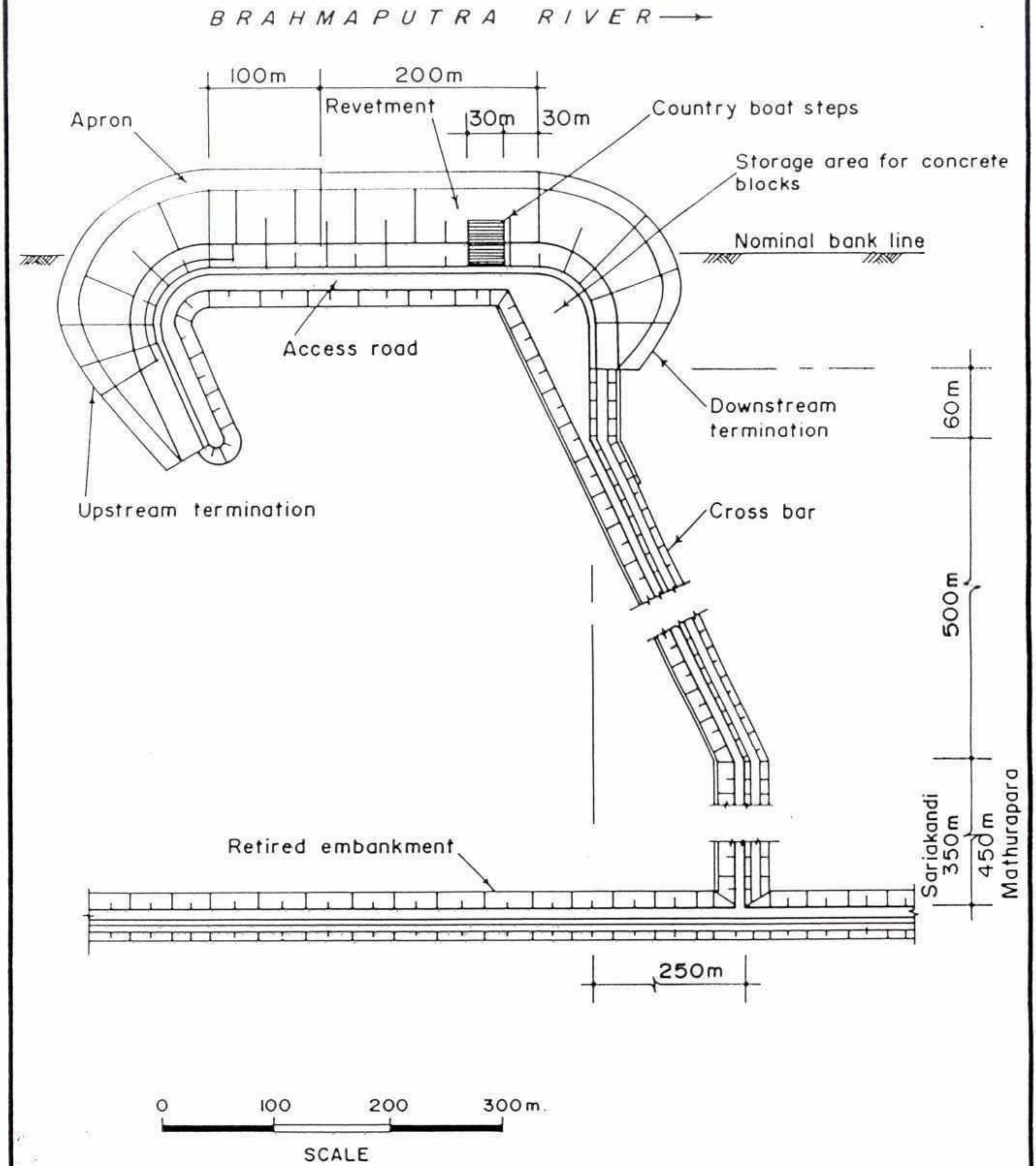


NOTES

1. Armouring not shown
2. Levels are in metres above an arbitrary datum.

BRTS Slope Stability Analysis: Revetment Profiles after Scour

	Annex 4
Halcrow/DHI/EPC/DIG *May 1993*	Figure 3.4



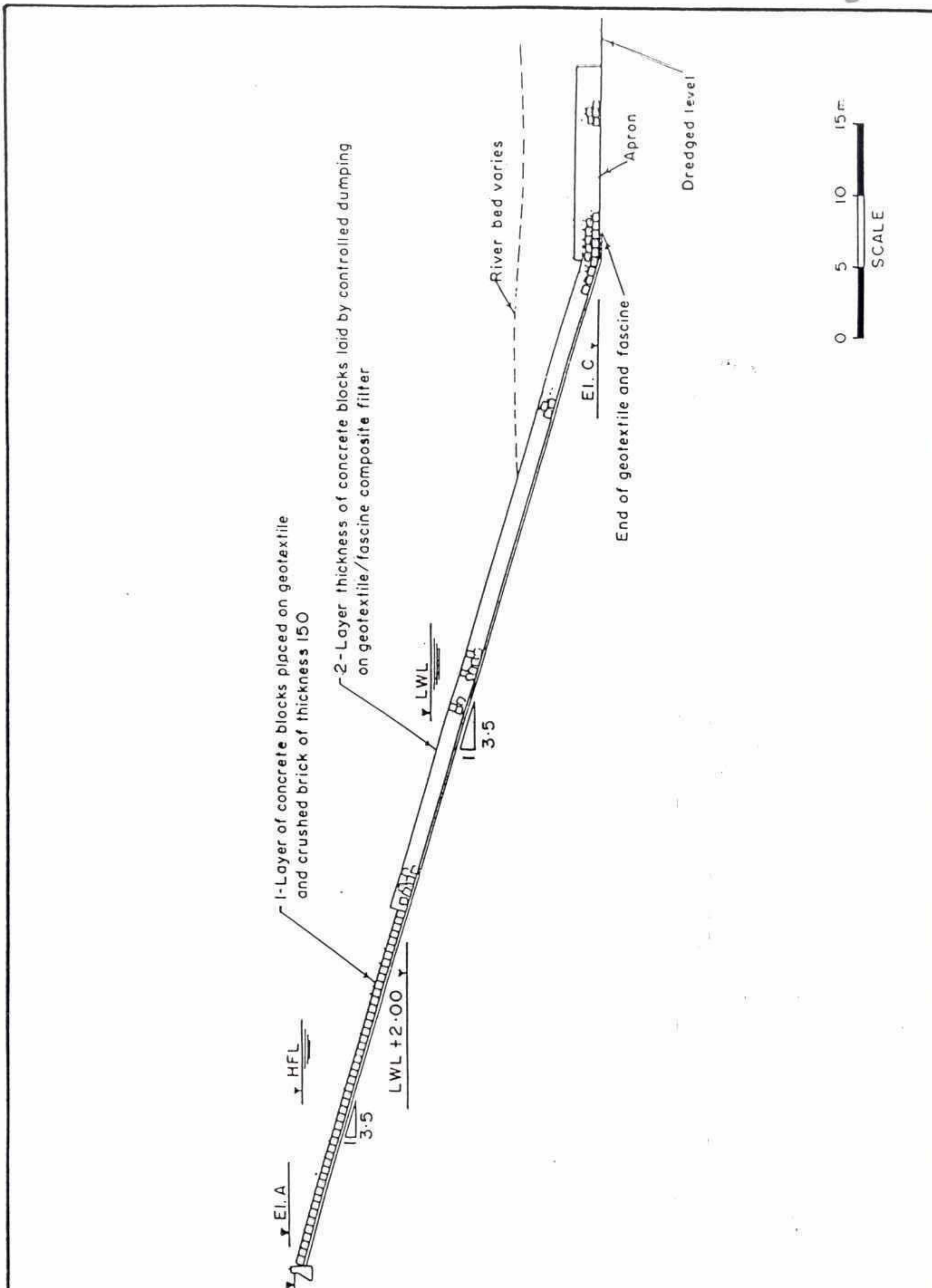
Typical Hard-Point and Cross-Bar

Halcrow/DHI/EPC/DIG

May 1993

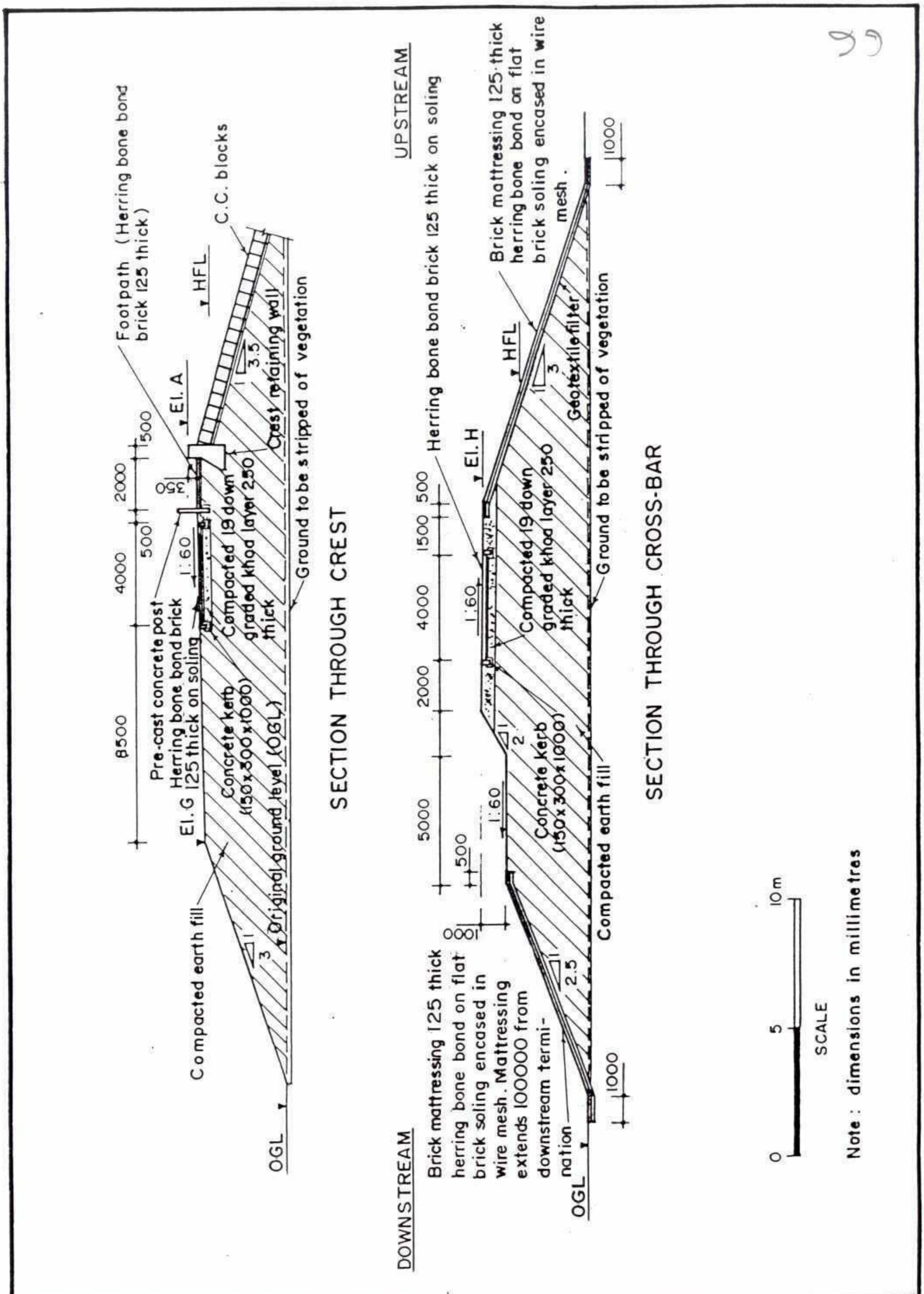
Annex 4

Figure 3.5



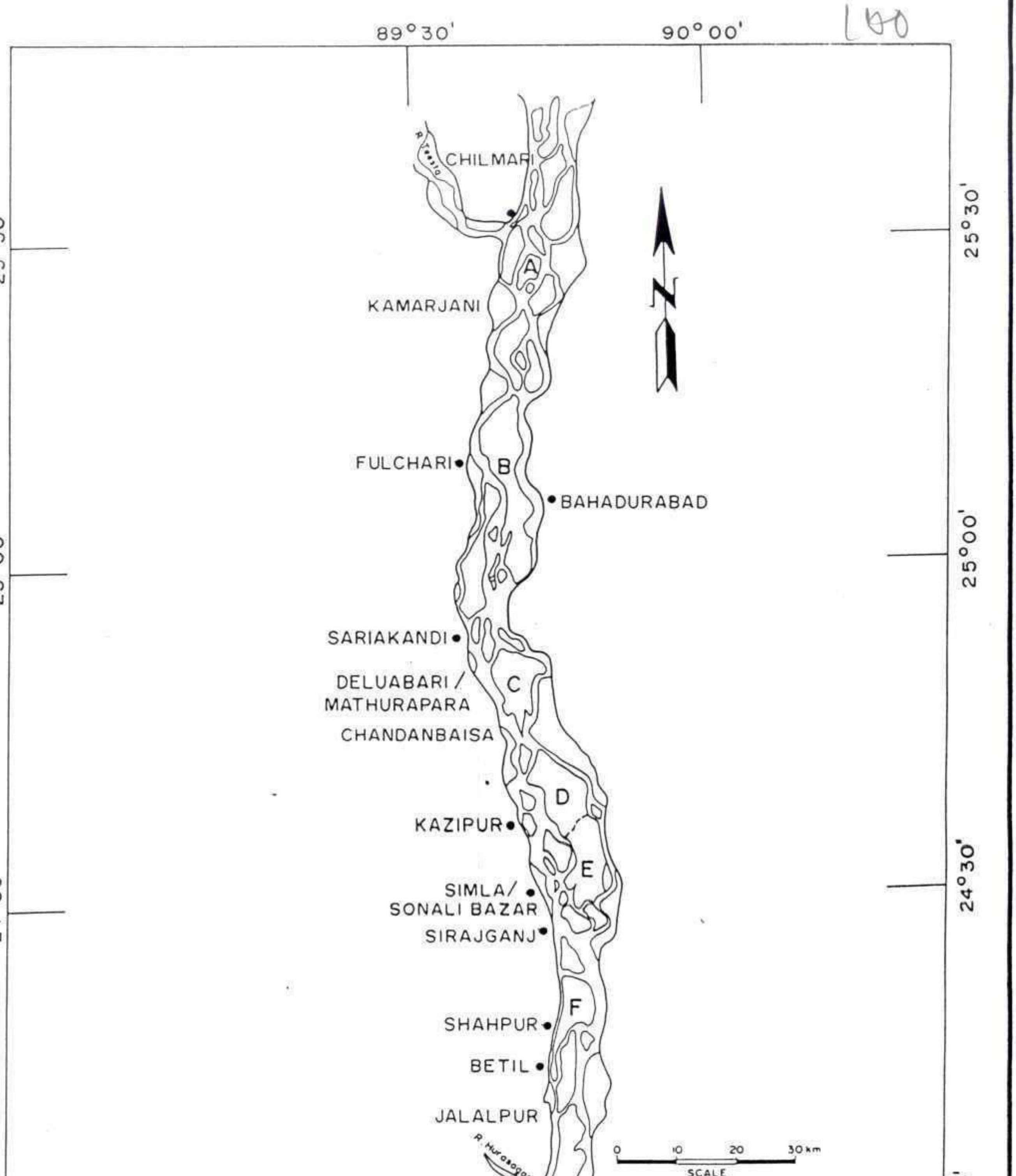
Typical Revetment Section

		Annex 4
Halcrow/DHI/EPC/DIG	*May 1993*	Figure 3.6



Crest and Cross-Bar Sections

	Annex 4
Halcrow/DHI/EPC/DIG May 1993	Figure 3.7



Present Island Pattern

		Annex 4
Halcrow/DHI/EPC/DIG	May 1993	Figure 4.1

Year	1994			1995				1996				1997		
Month	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS
Mobilisation		■■■												
Reclaim work area			■■■	■■■										
Set up Yards and Accommodation				■■■	■■■									
Stockpile armour					■■■	■■■	■■■	■■■	■■■	■■■	■■■			
Sections A and C:														
recover cc blocks							■■	■■						
dredge and fill							■■■	■■■						
redredge to profile							■■	■■■						
place geotextile							■	■■■	■					
place apron							■	■■■	■					
lower slope armouring							■	■■■	■					
upper slope armouring								■■■	■■					
crest works								■	■■					
Section D:														
recover cc blocks											■■	■■		
dredge and fill											■■■	■■■		
redredge to profile											■■	■■■		
place geotextile											■	■■■	■	
place apron											■	■■■	■	
lower slope armouring											■	■■■	■	
upper slope armouring												■■■	■■	
crest works												■	■■	
Section B:														
land reclamation					■■■				■■■					
demobilisation and clear up site													■■■	

Construction Programme for Works at Sirajganj

Halcrow/DHI/EPC/DIG *May 1993*

Annex 4
Figure 5.1

W2

Year	1994			1995				1996				1997		
Month	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS
Mobilisation		■■■												
Prepare work area			■■■	■										
Set up Yards and Accommodation				■■■	■■									
Stockpile armour					■■■	■■■	■■■	■■						
Kalitola, Sariakandi														
recover cc blocks							■■	■						
dredge and fill							■■■	■■■						
redredge to profile							■■	■■■						
place geotextile							■	■■■	■					
place apron							■	■■■	■					
lower slope armouring							■	■■■	■					
upper slope armouring								■■■	■■					
crest works & cross-bar								■	■■					
Mathurapara														
dredge and fill							■■■	■■■						
redredge to profile							■■	■■■						
place geotextile							■	■■■	■					
place apron							■	■■■	■					
lower slope armouring							■	■■■	■					
upper slope armouring								■■■	■■					
crest works & cross-bar								■	■■					
demobilisation and clear up site									■■■					

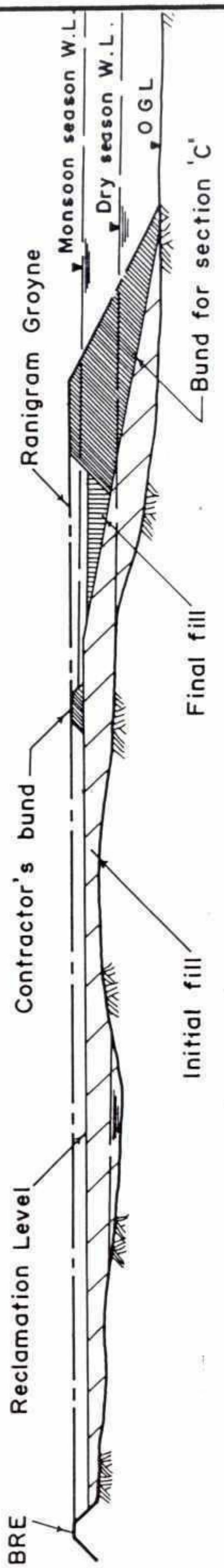
Construction Programme for Works at Sariakandi & Mathurapara

L03

Year	1			2				3				4		
Month	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS
Mobilisation		■■■												
Prepare work area			■■■	■										
Set up Yards and Accommodation				■■■	■■									
Stockpile armour					■■■	■■■	■■■	■■						
Hard-point 1 ------------ dredge and fill							■■■	■■■						
redredge to profile							■■	■■■						
place geotextile							■	■■■	■					
place apron							■	■■■	■					
lower slope armouring							■	■■■	■					
upper slope armouring								■■■	■■					
crest works & cross-bar								■	■■					
Hard-point 2 ------------ dredge and fill							■■■	■■■						
redredge to profile							■■	■■■						
place geotextile							■	■■■	■					
place apron							■	■■■	■					
lower slope armouring							■	■■■	■					
upper slope armouring								■■■	■■					
crest works & cross-bar								■	■■					
demobilisation and clear up site									■■■					

Construction Programme for Phase 1B & Phase 1C Works

104



SIRAJGANJ RECLAMATION

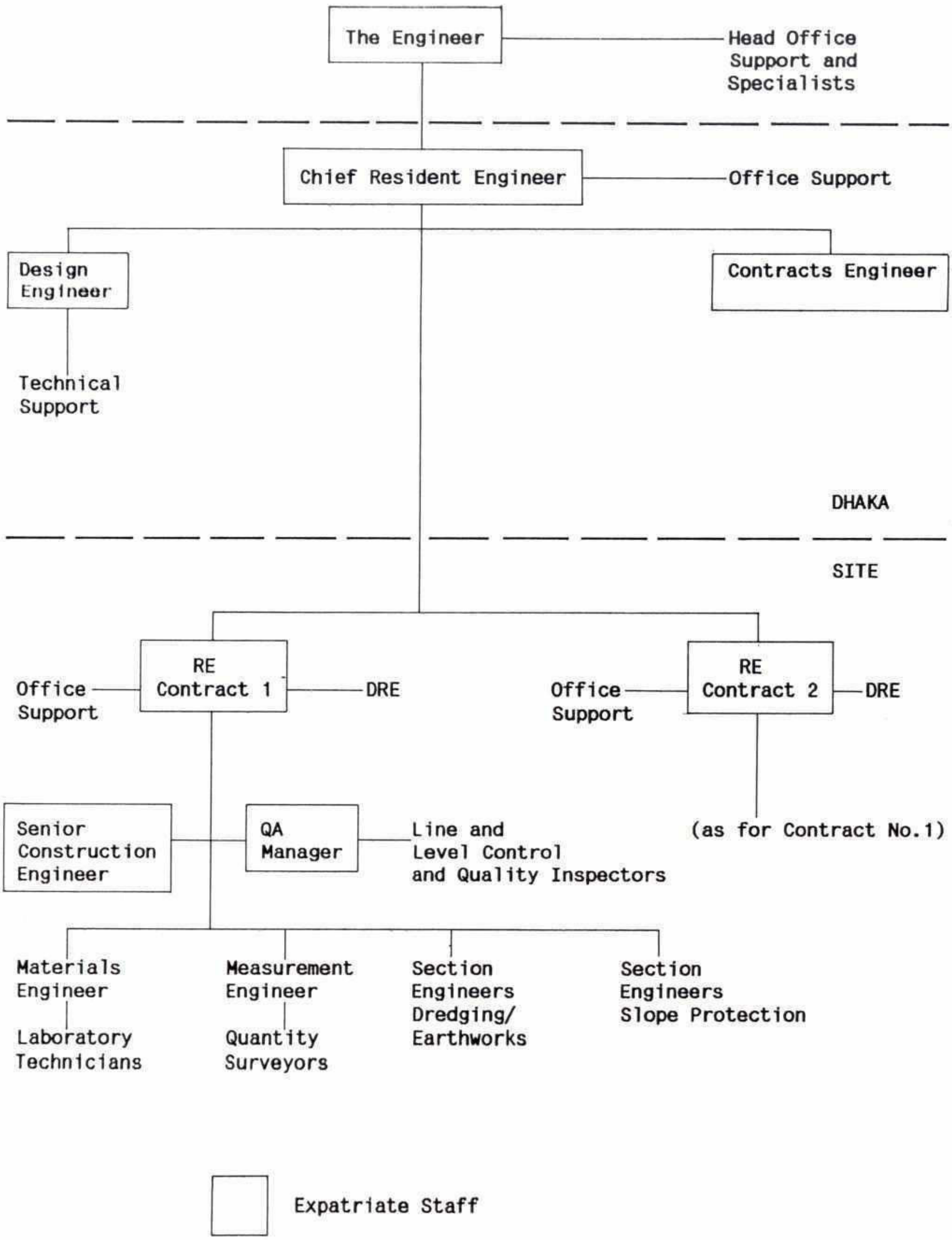
SECTION PARALLEL TO RANIGRAM GROYNE

N.T.S.

Sirajganj Reclamation - Section B

		Annex 4
Halcrow/DHI/EPC/DIG	*May 1993*	Figure 5.4

105



Construction Supervision: Staff Organogram

Annex 4

Year	1994			1995				1996				1997			P & T		Others	
Month	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	L	E	L	E
Chief Resident Engineer		II	III	III	III	III	III	III	III	III	III	III	III	III		38		
Design Engineer				III	III	III	III			III	III					18		
Assistant Design Engineer				III	III	III	III	III	III	III	III	III			27			
Assistant Design Engineer						III	III	III	III						12			
Draughtsmen				III	III	III	III	III	III	III	III	III			27			
Contracts Engineer			II				II	III	III	III	I		III	III		20		
Office Manager		II	III	III	III	III	III	III	III	III	III	III	III	III			38	
Office Support Staff		II	III	III	III	III	III	III	III	III	III	III	III	III				
															68	74	38	

Construction Supervision: Supervision-in-Chief

L07

Year	1994			1995				1996				1997			P & T		Others	
Month	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	L	E	L	E
Resident Engineer			III	III	III	III	III	III	III	III	III	III	III	III		36		
Deputy Resident Engineer			III	III	III	III	III	III	III	III	III	III	III	III	36			
Senior Construction Engineer						II	III	III	I	II	III	III	I			18		
Materials Engineer				III	III	III	III	III	III	III	III	III	III		30			
Measurement Engineer						I	III	III	III	III	III	III	III	III	19			
Section Engineers (up to max of 3)				III 2	III 2	III 3	III 3	III 3	III 2	III 1	III 3	III 3	III 2	III 1	75			
Diving Inspector				II		I	III	III	I	I	III	III	I	I	20			
Laboratory Technicians (up to max of 3)				III 1	III 2	III 2	III 3	III 3	III 2	III 1	III 3	III 3	III 2	III 1	69			
Topographical Surveyors (up to max of 2)			III 1	III 1	III 1	III 1	III 2	III 2	III 1	III 2	III 2	III 1	III 1		45			
Hydrographic Surveyors (up to max of 2)			III 1	III 1	III 1	III 1	III 2	III 2	III 1	III 1	III 2	III 2	III 1		45			
Quantity Surveyors (up to max of 2)						I 1	III 2	III 2	III 2	III 1	III 2	III 2	III 1	III 1	40			
QA Manager				III	III	III	III	III	III		III	III	III			27		
Works Inspectors (up to max of 3)				III 2	III 2	III 2	III 3	III 3	III 1	III 2	III 3	III 3	III 1		64			
Office Manager			III	III	III	III	III	III	III	III	III	III	III	III			36	
Draughtsmen (up to max to 2)			III 1	III 1	III 1	III 2	III 2	III 2	III 2	III 1	III 1	III 1	III 1	III 1	46			
Technical Support Staff			III	III	III	III	III	III	III	III	III	III	III	III				
Office Support Staff			III	III	III	III	III	III	III	III	III	III	III	III				
Visiting Specialist							I	I I			I	I I				6		
															489	87	36	

Construction Supervision: Sirajganj Contract

Annex 4

Halcrow/DHI/EPC/DIG *May 1993* Figure 7.3

108

Year	1994			1995				1996				1997			P & T		Others	
Month	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	OND	JFM	AMJ	JAS	L	E	L	E
Resident Engineer			III	III	III	III	III	III	III	III						24		
Deputy Resident Engineer			III	III	III	III	III	III	III	III					24			
Senior Construction Engineer						II	III	III	I							9		
Materials Engineer				III	III	III	III	III	III						18			
Measurement Engineer						I	III	III	III	III					13			
Section Engineers (up to max of 6)				III 2	III 2	III 3	III 6	III 6	III 3	III 1					69			
Diving Inspector				II		I	III	III	I	I					11			
Laboratory Technicians (up to max of 6)				III 1	III 2	III 4	III 6	III 6	III 2	III 1					66			
Topographical Surveyors (up to max of 3)			III 1	III 1	III 1	III 1	III 3	III 3	III 2	III 1					39			
Hydrographic Surveyors (up to max of 3)			III 1	III 1	III 1	III 2	III 3	III 3	III 1						36			
Quantity Surveyors (up to max of 4)						I 1	III 4	III 4	III 3	III 1					37			
QA Manager				III	III	III	III	III	III							18		
Works Inspectors (up to max of 6)				III 2	III 3	III 4	III 6	III 6	III 1						64			
Office Manager			III	III	III	III	III	III	III	III							24	
Draughtsmen (up to max to 2)			III 1	III 1	III 1	III 2	III 2	III 2	III 2	III 1					34			
Technical Support Staff			III	III	III	III	III	III	III	III								
Office Support Staff			III	III	III	III	III	III	III	III								
Visiting Specialist							I	I I								3		
															411	54	24	

Construction Supervision: Sariakandi and Mathurapara Contract

109

APPENDICES

HALCROW

APPENDIX A

RANKING OF SITES (APRIL 1991)

BRAHMAPUTRA RIVER TRAINING STUDIES
SELECTION OF LOCATIONS FOR PRIORITY WORKS

1. BACKGROUND

Four locations have to date been selected for the preparation of detailed designs in accordance with the BRTS terms of reference. These are Fulcharighat, Serajganj, Sariakandi and Kazipur. It is necessary as a matter of urgency to select two more for the first set of detailed designs that are to be prepared by the end of August 1991.

On 13 March a proposal for this selection was presented to a meeting held at BUET attended amongst others by Prof Nishad, Dr Shahjahan and Mr Rafiquzzaman, Director Planning (General). There was general agreement with the arguments as presented but it was agreed that a final decision should be deferred until the matter could be reviewed by Dr T Herman, the project Task Manager from the World Bank, Washington, who was expected to visit Dhaka in the near future. The following note sets down the basis on which the presentation was made.

2. THE SHORTLIST

Setting aside the locations already selected, those that now remain on the shortlist are:

Chandanbaisa
Betil
Kamarjani
Mathurapara
Sonalibazar
Jalalpur.

The order presented here corresponds to that suggested by Dr Herman in his memorandum dated 3 January 1991.

The anabranch bends at both Sonalibazar and Jalalpur have very high radius to width ratios and the heavy shoaling that has occurred is a strong indication that chute cutoffs may be expected to form during the coming monsoon season, resulting in a major reduction in the rate of bank erosion. This being the case, the locally retired embankment that has been constructed at Jalalpur and the one that will shortly be completed by the BWDB at Sonalibazar should therefore have an acceptably long life expectancy.

The four remaining locations have therefore been reassessed in the light of the outcome of morphological studies undertaken during the past three months and other information that has become available. In the following section the four sites are considered in order from north to south.

112w

3. MORPHOLOGICAL AND OTHER CONSIDERATIONS

3.1 Kamarjani

Kamarjani is a small but active jute bazaar, reportedly of long standing, situated on the reach of the Brahmaputra between the Teesta confluence and where the Old Brahmaputra river distributary takes off. Over the past 30 years there has been relatively little change in the banklines in this reach and the braid intensity appears to have remained more or less the same. There is a suggestion that the influence of the Teesta has tended, at least in the last decade, to deflect the main anabranches away from the right bank over a length extending downstream of Kamarjani. The southern bank of the Teesta confluence itself is relatively stable. The current erosion at Kamarjani appears therefore to be associated with a minor anabranch bend that is migrating downstream without strong lateral development. If this trend continues then there is unfortunately a high probability that the bazaar of Kamarjani will be destroyed within the coming season and that the BRE will be breached at this point.

Upstream of Kamarjani the original BRE remains intact and from some 1000m north of the bazaar the distance between embankment and the present bank line is generally more than 300m. Unless there is a change in the river's behaviour, this separation should be sufficient to prevent the BRE from being threatened for the next 10 to 20 years.

Downstream of the village the BRE has been retired four times over a length of about 3 km and is now on average about 400m back from the bank line. If the present anabranch bend continues to migrate downstream and maintain its plan geometry then it is likely erode up to 1200m of this section of the BRE. Beyond this point, seems probable that there would be no further damage to the embankment.

Following this argument, there would therefore seem to be a case where construction of a locally retired embankment of less than 3km in length would provide a reasonable level of security. Conversely, if the bank in the vicinity of Kamarjani bazaar were to be stabilised by revetment over a distance of, say, 1,500m, there is the possibility that this would encourage bank erosion upstream and downstream of the revetment, in accordance with the "hard point" theory. This in turn would threaten substantial sectionsof the existing BRE.

An investment in revetment (or other stabilisation technique) at Kamarjani should therefore be seen as primarily providing protection to the bazaar alone. The value of the property comprising the bazaar has been estimated at 3.4 crore taka and that in additional land worth 6.5 crore taka could be lost, some of which would still be lost with the protection works. The cost of the works on the other hand has been estimated at 30 crore taka.

3.2 Mathurapara

Mathurapara represents the current downstream limit of a reach of relatively high erosion extending down from Sariakandi and associated with a large radius bend with a chord length of about 8 km. This reach includes the site of the bazaar of Deluabari which has now been totally destroyed. The geomorphological studies have drawn attention to the fact that the reach of river between Sariakandi and Serajganj has, within the study area, experienced the most severe right bank erosion during the past 30 years and that the macro-morphological characteristics of the river's high flow channel are such that it would be reasonable to expect that this westward

113

3.3 Chandanbaisa

Chandanbaisa is an active permanent bazaar situated about 8 km downstream of Mathurapara. It has many features in common with Kamarjani both from the point of view of the property under threat and the form of the bank erosion process, although the value of the former has been estimated at about twice that of Kamarjani. The proximity of the settlement to the river and the aggressiveness of the bank attack means that unhappily it is highly probable that most if not all of the buildings will be destroyed during the coming monsoon season.

The main difference between the two sites in terms of morphology is that Chandanbaisa is situated on a particularly unstable section of the river. The long term tendency for the river to erode its right bank in this area has been described in the previous sub-section. The reach that culminates in the Chandanbaisa bend exhibits characterisitcs that are not typical of the Jamuna section of the Brahmaputra as a whole and the inference from this is that further major erosion must be expected but that it will be unusually difficult to predict where it will occur and how it will develop. For his reason it will be necessary to follow a more conservative approach when determining the overall length of the revetment than would otherwise be the case.

Nonetheless, it is quite possible that if an investment were made in a length of revetment in the vicinity of Chandanbaisa, where the erosion was at its most severe at the time of construction, then in a short space of time the focus of erosion could switch to a different stretch of the river. Although the hardened section would almost certainly perform a useful function again in due course, its efficacy would be less than optimal and in economic terms the effective present value would be heavily discounted.

In short, in hard economic terms, it would probably be more cost effective to wait until the river has settled down somewhat in this region before making major investments in stabilisation works. The probable destruction of the existing settlement within the coming one or two seasons would strengthen this argument.

3.4 Betil

Betil is situated on the lower reach of the Jamuna, about 20 km upstream of the Hurasagar confluence. It is the riverside extremity of a heavy concentration of small handloom businesses that extends up to the Bera-Bogra highway.

This reach of the river is south of the Jamuna Bridge site and exhibits characteristics that are quite distinct from those of the Mathurapra-Chandanbaisa reach. A point in common is that since 1964 the bank line has moved westward by about 2 km and the macro-morphological studies indicate that there is a likelihood that erosion of the right bank will continue in this vicinity but probably at a rather lower overall rate. The influence of the Hurasagar and the Ganges confluence further downstream have yet to be assessed, as also has the impact of construction of the bridge.

At present the bank erosion is associated with a minor anabranch that hugs the right bank while the major branch holds to the left bank. There is the possibility that with the switching of anabranch dominance from the left to the right upstream of the bridge site there may be an associated tendency for the sinusoidal planform to reassert itself resulting in an trend will continue and could occur at an average rate overall of the order of 100m per year. Localised embayment type erosion could of course develop at a much higher rate.

This large scale tendency is perhaps accentuated at the present time by the apparent switch of the main anabranch system from left to right banks around 1984. It may be that as a consequence of this switch the anabranch waveform in this area is less stable than normal, resulting in a less well defined erosion pattern.

At its closest, the Bangali river is now only separated from the Brahmaputra by about 1.5 km and at the apex of the bend the separation is a little over 2 km. Were the bend to tighten its radius it could enter the rapid development phase which in other cases has resulted in reported bank erosion rates of up to 700m a year. If no action is taken it would therefore seem probable that within a matter of years, and maybe within as little as 3 years, the Brahmaputra could break through into the Bangali river. The consequences of such a breakthrough are hard to quantify but there is a probability that some significant degradation of the Bangali bed would occur and that this could lead to a systematic increase in its conveyance. The worst case scenario would see the Bangali river developing into a major distributary which in time could become a secondary course of the Brahmaputra river itself. This would clearly have far reaching consequences.

Even before such a breakthrough occurs there would be serious problems over maintaining the BRE. The narrow strip of land between the two rivers would be the only practicable alignment for the embankment and it would be very vulnerable to attack from both sides.

Downstream of Mathurapara the BRE has a long history of retirement with muliple realignments taking place over a reach of at least 10 km. If the Mathurapra eroding bend follows the pattern of migrating downstream, it will progressively eat into the realigned embankment and create a chronic maintenance problem for years to come.

Thus although in this case there is little of high value in the immediate vicinity of the bankline to protect, the long term integrity of the BRE is under serious threat and there is a strong possibility that the Brahmaputra could break through to the Bangali river. Even before a breakthrough it will become increasingly difficult to maintain the BRE and the scope for retirement is strictly limited by the presence of the Bangali river; this will mean that breaches will be a common occurrence and the proximity of the Bengali will ensure that the flow through the breach is transferred relatively easily to downstream areas, creating a large affected area. The frequency of breach flows will in itself increase the probability of a permanent connection developing between the two rivers.

If on the other hand a hard point can be established at Mathurapara in conjunction with the hardening of Sariakandi then there is a good probability that the bend in between will reach its maximum westward movement and back out again before there is a serious likelihood of a breakthrough into the Bengali developing into a serious threat. Such a hard point could consist of a stretch of revetment with well anchored upstream and downstream limits to resist any outflanking movement. The cost has been estimated to be of the order of 25 crore taka.

115

increase in the right bank anabranch conveyance downstream. Were this to occur, the attack in the vicinity of Betil would probably become more severe.

A short distance downstream of Betil the same anabranch has over the past 10 years eroded an unusually deep embayment at Jalalpur. The maximum rate of retreat occurred between 1983 and 1989 with a total lateral movement of 1.2 km and relatively little downstream migration of the bend. This extreme case of bend erosion appears to have reached its climax and there are strong indications that a chute cutoff is developing. The influence of this cutoff on the infant Betil bend is a matter of conjecture but one consequence would seem to be that any constraint on downstream migration of the Betil bend that was due to the Jalalpur bend will now be removed.

The present phase of the Betil bank erosion started in 1989 and appears to be developing in intensity, particularly at the downstream end where the ratio of bend radius to channel width is decreasing.

Based on the behaviour of other bends, it seems likely that the Betil bend will move both laterally and in a downstream direction, with the former component being the larger. Such a development would over a period of perhaps 5 or 6 years displace at least as many people as at Kamarjani and Chandanbaisa together with the disruption to an important section of the handloom industry. As the erosion migrated downstream the BRE would be progressively breached, resulting in direct overbank flooding of the hinterland with its concentration of handloom installations. The impact of such breach flows will at certain times of the year be substantial due to the impeded drainage resulting from the backwater effects up the Hurasagar river.

By constructing a length of revetment towards the lower end of the bend where the erosion is most active, much of the population concentration could be protected. This hardening should also prevent the bend from migrating downstream and thus reduce the likelihood of the BRE being breached in this direction. There would still be no constraint on the lateral development of the bend initially but the presence of the hard point should limit the maximum extent of the erosion and with the alignment of the locally retired embankment proposed by the BWDB the risk of a further breach upstream should be small. As a longer term measure, this hard point would be the first of a series aimed at stabilising the westward drift of this lower limb. For this purpose it is sensibly placed in relation to the proposed Jamuna Bridge site. The cost of the works have been estimated at about 30 crore taka.

3.5 General Comments

The foregoing descriptions of the four sites are based on the data that is available at this time and since the purpose of the study is to generate new information it is reasonable to expect that some of the tentative predictions will be modified. However the morphological and other studies have now progressed to a point where some real insight into the river behaviour is emerging and thus the comparative assessment of the sites presented here is on a considerably firmer base than that of the December 1990 discussion paper.

The type of bank stabilisation works envisaged for each location are broadly similar in form, consisting of a varying length of conventional, but carefully designed, revetment with returned ends to guard against outflanking. The length in each case will be determined by consideration

116

of the property or other investment that may be protected, the extent of the existing deep scour, the anticipated movement of the scour trench and the consequences of upstream and downstream bend scour development.

The cost estimates for the works remain very tentative and should be taken as values for comparison only. They are almost certainly on the low side and may be expected to increase by the order of 25% once all aspects of the design have been fully quantified.

For this exercise the social aspects have not been requantified because within the timescale of the implementation of any works the situation will certainly have changed so dramatically that an assessment based on the current situation is certain to be misleading. Given the predictions as to the development of the bank erosion and its consequences in the short term, it is in any case unlikely to be a deciding factor. If anything the weighting would tend to be towards Betil because the expectation is that there the rate of erosion may be such that population displacement will be more attenuated and still an issue at the time of implementation.

Similarly the value of land lost and agricultural disbenefits have not been reassessed since there has been no change in the quality of the database since the December 1990 discussion paper and earlier sensitivity tests showed that the ranking was relatively insensitive to these parameters.

4. CONCLUSION

The estimated cost of the bank stabilisation works at each of the four locations is broadly of the same order, with that at Chandanbaisa somewhat higher due to the uncertainty as to the behaviour of the scour pattern and that at Mathurapara marginally lower because the principal objective is to form a hard point and not to protect any specific property or installation.

By looking at the benefits to be derived it is apparent that Kamarjani offers the least return to the investment of the four, given that a locally retired embankment will be constructed to maintain the continuity of the BRE.

At the other end of the scale, the benefits to be derived from stabilising the bank of the Jamuna, where it is within a comparatively short distance of the Bengali river, are now clearly seen to be substantial, although hard to quantify at this stage of the FAP programme. This will move Mathurapara firmly into the top position out of the four.

The choice between Chandanbaisa and Betil is more finely balanced. The principal deciding factor is that the behaviour of the river is more uncertain at Chandanbaisa and therefore that any investment will have a lower probability of useful return, particularly in present value terms. A significant contributary argument is that at the time of probable works implementation the direct impact of bank stabilisation measures in terms of protection of property and minimisation of population displacement will be greater at Betil than at Chandanbaisa.

These arguments have been presented in the form of a simple decision, matrix which is attached, from which it can be seen that the recommended order for priority of investment is:

117

1. Mathurapara
2. Betil
3. Chandanbaisa
4. Kamarjani

April 1991

PRIORITY WORKS RANKING: DECISION MATRIX
(FOR RIVER POSITIONS 5 TO 8)

LOCATION	ISSUE			
	(1) Length of BRE under threat	(2) Will a single BRE retirement provide a permanent solution ?	(3) Influence of protection works on macro - stability of the reach	(4) Overall Cost - effec- tiveness
Kamarjani	□	□	□	◪
Mathurapara	■	■	■	■
Chandanbaisa	■	■	◪	□
Betil	■	◪	◪	■

Key : (1) □ less than 2 km

◪ 2 to 5 km

■ greater than 5 km

(2) □ likely

◪ uncertain

■ unlikely

(3) □ protection measures too localised to influence macro stability

◪ measures may have some influence on macro stability

■ measure likely to an influence on macro-stability

(4) □ worse (higher risk or lower damage avoided)

◪

■ better (lower risk or higher damage accorded)

APPENDIX B

GEOLOGY AND TECTONICS OF THE STUDY AREA

APPENDIX B

GEOLOGY AND TECTONICS OF THE STUDY AREA

1. TECTONIC FRAMEWORK

In plate tectonic theory, the Bengal Basin forms part of the Indo-Australian plate which is moving north east and is drawn beneath the Eurasian plate into a subduction zone, at a consuming margin. The plate boundary is assumed to run along the Indus/Tsangpo Suture Line, to the north of the Himalayan mountain chain (Gansser, Ref. 18).

The collection of the plates has resulted in crustal shortening. Jhingram (Ref. 19) gives the average rate of northward drift of the Indo-Australian plate since the end of the Cretaceous period (65 million years ago) as 80mm/year. The Indian plate consists of the crystalline basement, exposed in places but covered elsewhere by sediment, as in the Bengal Basin.

The two main tectonic components of the Bengal Basin are the stable shelf which forms part of the Indian Platform area to the northwest; and a subsiding geosyncline to the southeast. The dividing line between the two is known as the Hinge Zone, passing approximately through Calcutta, Kushtia and Mymensingh evidence which might exist is masked by the very recent flood plain sediments. Therefore only inferential geophysical evidence exists that these faults continue as lineaments under cover of alluvial deposits to intersect near the proposed barrage site. The evidence for the existence of and activity on these faults is as follows:

- A horizontal movement of some 100 mm along the Dhubri fault took place during the 1930 Dhubri earthquake, in a locality some 50 km north of barrage site 'A' (MacDonald, Ref. 20).

- The Mymensingh fault is a vertical fault with downthrow to the northeast and the fault scarp marks the northeastern limit of the elevated tract of the Madhupur forest. Fergusson (Ref. 35) postulated sudden uplift along this fault during the major Bengal earthquake in 1762. Other authors (Morgan, Ref. 21) assume that a slow creeping movement is more probable. The visible scarp of the Mymensingh fault is some 40 km south east of barrage site 'A'.

The Dauki fault forms the southern limit of the Shillong plateau which rises abruptly form the plains of the Bengal and Sylhet basins. It is assumed to be a very major dextral tear fault, with a possible horizontal movement of 300 km since the Miocene (past 10 million years) era and a vertical throw to the south. Kailasam (Ref. 22) states "The Sylhet basin to the south a know to have subsided to a depth of 10 to 15 km during the past hundred years" but does not quote the evidence. Geodetic levelling carried out by the Survey of India indicates a rise of the Shillong Plateau of 25 mm only in the period 1910 to 1977 (Ref. 23). The localities where movements of the Dauki fault take place are some 200 km east of the barrage site.

The various other potentially active faults with the same alignments as the three megalineaments described above are indicated by aeromagnetic surveys. The typical spacing of the assumed basement faults of all three families is 10 km.

Geomorphological studies suggest that the present-day Brahmaputra follows a line of relative subsidence, with areas of uplift situated to the east and west of it.

The only known tectonic discontinuity affecting barrage site 'B' is a north-south lineament following the line of the Brahmaputra, which may be a continuation of the Dhubri Fault. There are no other known faults or discontinuities in the immediate proximity of barrage site 'B'. The potentially active faults affecting barrage site 'A' pass 40 km north (Dauki Fault) and 20 km east (Mymensingh Fault) of this site.

The geophysical evidence on the tectonic structure of the basement, obtained during mineral resources surveys, is summarised on figure 3.1.

Barrage site 'C' on the Ganges lies just to the north of the "Hinge Zone", whilst Barrage site 'D' lies over the Hinge Zone. The Hinge Zone may be a tectonic zone of weakness, but there is no seismological or geomorphological evidence of any recent activity along it. According to the tectonic interpretation of the airborne geophysical data presented on figure 3.1, a fault or a basement discontinuity with a north-west to south-east trend pointing towards barrage site 'C' terminates just to the south-east of it. There is no surface manifestation of this discontinuity. No faults, active or otherwise, are known to exist in the area of barrage site 'D'.

2. GEOLOGY

The stratigraphic table for the Bogra section given below is taken from a recent publication of the Geological Survey of Bangladesh (Ref. 37).

Age		Lithology
Recent to Sub-Recent Alluvium		Sand, silt and clay
	UNCONFORMITY	
Pleistocene	Madhupur Clay	Clay, sandy clay, yellow -brown sticky.
	UNCONFORMITY	
Early Miocene	Jamalganj Formation	Fine to medium grained sandstone, sandy and silty shale, siltstone, shale.
Oligocene	BograFormation	Siltstone,carbonaceous shale and fine grained sandstone.
	UNCONFORMITY	
Late Eocene	Kopili Formation	Sandstone, locally glauconitic and highly fossiliferous, shale with thin calcareous bands.
Middle to late Eocene	Sylhet Limestone Formation	Numulitic limestone with interbeddedsand-stone.
Middle Eocene to Late Cretaceous	Tura Sandstone Formation	Gray and white sandstone, with subordinate, greenish grey shale and coal.
	UNCONFORMITY	
Late Cretaceous	Sibganj Formation Trapwash	Coarse yellow brown sandstone, volcanic material, white clay
	UNCONFORMITY	
Late Jurassic to Middle Cretaceous	Rajmahal Trap	Amygdaloidal basalt Serpentinized shale
(Upper Gondwana Group)		and agglomerate
	UNCONFORMITY	
Late Permian (Lower Gondwana Group)	Paharpur Formation	Feldspathic sandstone, shale and coal beds.
	Kuchma Formation Barakar Group	Sandstone and grit with subordinate shale interbedded with coal.
	UNCONFORMITY	
Precambrian	Basement Complex	Gneiss, schist, Granodiorite, Quartz diorite.

123

The basement complex consists of gneisses, intruded by granodiorites and quartz diorites. It is unconformably overlain by coal bearing measures consisting of sandstone and grit with subordinate shales and interbedded with coal seams. Five coal seams with thickness of 6m, 22m, 4m and 6m are present in the type locality of Jaipurhat. The Lower Gondwana Group is unconformably overlain by basaltic Rajmahal Trap of the Upper Gondwana Group. The trap rocks are mainly hornblende basalt, olivine basalt and andesite. The Sibganj Formation unconformably overlies and Rajmahal Trap and consists of red, ferruginous shale, claystone and green and red, ferruginous, course sandstone, derived from weathering of the basalts ("Trapwash").

The overlying Tura Formation consists of sandstone with subordinate shale and (fossiliferous) marl. The Tura Sandstone is followed by the Sylhet limestone, a prominent marker horizon. The limestone is grey or brownish, massive and hard. The Kopili formation is, locally, a grey, silty, carbonaceous, pyritic shale and glauconitic sandstone. The overlying Bogra Formation consists of alternations of shale and sandstone and claystone. This is followed by the Dupi Tila Formation composed of pebble beds, grit beds, coarse to fine sandstone and clay shale. The Dupi Tila Sandstones are sufficiently poorly consolidated to be logged as "sands" in many groundwater investigations.

The Madhupur clay lies unconformably over the Dupi Tila Formation. It consists of mottled clay interbedded with sand, and silt with ferruginous and calcareous nodules. The Madhupur Clay is succeeded by Recent and Sub-Recent alluvium deposits, comprising sand, silt and clay with occasional gravel.

Over the entire Bogra slope the dips are to the south east, very gentle (one degree or so) between the crest of the Rangpur Saddle and Bogra, and somewhat steeper (3 to 4 degrees) from Bogra up to the Hinge Zone.

The entire succession described above is cut into a series of horsts and grabens by the three families of vertical faults described above.

East of the Hinge Zone the basement rocks dip very steeply towards the southeast, attaining an eventual depth of about 13000 m. The eastward extension of the Dauki fault marks the northern boundary of the basin, with the Shillong plateau to the north.

The Madhupur tract is an uplifted Pleistocene terrace, covered in jungle, extending from Dhaka in the south to near Jamalpur and Mymensingh in the north. It has an abrupt western margin, rising to an elevation of about 33 m SOB whence it slopes gently to the southeast to a level of 6m SOB. The western margin is formed by a series of 6 echelon faults, varying in length from 4 km to 8 km, upthrown to the east by between 6 and 18 m.

Surrounding the Madhupur forest are the floodplain deposits described in sub-section 3.1.

APPENDIX C

REVIEW OF REVETMENT DESIGN

125

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

A REVIEW OF THE BASIC PRINCIPLES BEHIND THE BRTS BANK REVETMENT DESIGN RELATING SPECIFICALLY TO THE BRAHMAPUTRA RIVER CONDITIONS

CONTENTS

BRAHMAPUTRA RIVER TRAINING STUDY

A REVIEW OF THE BASIC PRINCIPLES BEHIND THE BRTS BANK REVETMENT DESIGN RELATING SPECIFICALLY TO THE BRAHMAPUTRA RIVER CONDITIONS

1. INTRODUCTION

The following notes arise from a discussion held between Dr C A Fleming (Halcrow Director), Mr O J Jensen (BRTS Physical Modelling Specialist, DHI) and Mr A P G Russell (BRTS Team Leader, Halcrow) on 13 January 1992. The object of the meeting was to draw on the wide experience of Dr Fleming regarding the design of rock and concrete block slope protection to withstand both wave and flow velocity forces in many parts of the world.

The discussion focussed on the essential requirements that the revetment design must satisfy, the minimum permissible design criteria and the practicalities of construction.

The typical bank stabilisation unit, or hard point, was for this purpose considered as four elements: slope protection above LWL (Low Water Level), slope protection below LWL, the toe or falling apron, and the upstream and downstream terminations. The fourth element is composed of the first three but has a distinct function, is exposed to more adverse conditions and involves different construction techniques.

A common feature of all elements is the geotextile membrane which is seen as not merely a filter layer but in many respects the primary structural component. The role of the layer overlying the geotextile being to hold the membrane in position in intimate contact with the underlying soil and to provide protection against ultra-violet light (above LWL) and mechanical damage.

2. SLOPE PROTECTION ABOVE LWL

The distinctive feature of this element is that both the geotextile membrane and the protective/ballast layer, sometimes referred to as the armour layer, can be placed in the dry and using land based plant. Moreover the whole surface will be exposed for at least 200 days in a year and any damage can be readily seen and simply remedied.

It was concluded that the present BRTS design which specifies two layers of carefully placed concrete cubes over the geotextile membrane was unnecessarily conservative; the upper layer plays no clear role and if conditions are such that one layer is displaced then it is almost certain that the second layer will also be displaced (the results of the physical modelling support this thesis). The two layers are a thought to be a hangover from old Indian practice dating back to days before reliable filter materials were available. On this basis, a single layer would suffice provided that (a) adequate quality control during placing was achieved, (b) exposure of the membrane to ultra-violet light could be avoided and (c) timely maintenance could be guaranteed in the unlikely event that a block was displaced.

An alternative form for the protective/ballast layer that would satisfy the same functional requirements would be concrete blocks of the same or greater weight per square metre but with larger planform dimensions. Other systems could also be technically feasible but there was no precedent or experience of their use on the Brahmaputra and such alternatives are probably best left to be addressed by the FAP-21/22 pilot studies.

3. SLOPE PROTECTION BELOW LWL

This element of the works is seen as presenting the most significant problems in terms of both construction and maintenance. Since the whole face will be permanently submerged in turbid water, and in some cases after construction will be covered for much of the time by sediment, it will be extremely difficult to monitor its structural performance and to control both construction and maintenance. The design must give due consideration to both these aspects.

If it were possible to devise a system of placing concrete blocks underwater in a closely packed uniform pattern with a high level of confidence then a single layer of such blocks could theoretically meet the functionally requirements. However there is no experience of the performance of such an arrangement under the turbulent conditions found in the Brahmaputra at high flows and since full reliance would be placed on this single layer it would be prudent to increase the block size, by perhaps 20 percent (or weight by 20 percent?), in order to provide an enhanced factor of safety. There would thus be a significant reduction in concrete volume to be offset against increased cost of placing. The balance between these two factors would depend to a large degree on the contractor's access to suitable plant.

If the blocks could be linked into a mats then further reduction in concrete volume could be achieved at the expense of further investment in specialist plant for laying the mats. The ultimate development in this direction being a composite mattress composed of bonded blocks and membrane, which offers a very satisfactory engineering solution but requires a very high investment in specialist plant (the same system could of course be used above LWL).

Given that the precision placing is not feasible then the alternatives are dumped rock or dumped concrete blocks.

Although it might be argued that a single layer of dumped blocks could meet the functional requirements, in practice it would be impracticable to achieve a guaranteed single layer coverage under the prevailing working conditions. The absence of full coverage at critical locations such as laps in the membrane could result in rapid failure of the system. The specification of two layers of dumped blocks is consistent with worldwide practice for such conditions and provides a reasonable level of confidence that not less than a full single layer will be achieved over the whole area, given a level of placing control that is not excessive using modern plant and aids. The specification should reflect this concept and not be aimed at achieving complete coverage of two layers, which would imply the actual placement of perhaps 25 percent additional material over the nominal quantity.

There is no evidence a mix of block sizes will perform any better in such circumstances and there are significant disadvantages in terms of potential for segregation during dumping and added difficulty over distribution monitoring. Uniform block size is thus considered the better choice.

Graded rock instead of blocks offers an equally satisfactory technical solution and the choice may be made on the basis of cost and scope for the employment of unskilled labour; the former will in turn depend on material prices and the contractor's access to suitable plant.

4. TOE OR FALLING APRON

The primary function of the toe or falling apron is to prevent toe scour undermining the slope and initiating progressive geotechnical slope failure. As a secondary function it provides a toe weight which improves the slope stability.

The design of this element must be such that it deforms in response to scour so that the newly formed slope is adequately protected. The two principal forms that are commonly adopted for this purpose are the Indian Practice type launching apron and the US Corps of Engineer's type section, in which the material is concentrated in the toe. Both are known to have performed satisfactorily elsewhere but there is no specific information relating to the Brahmaputra River. The on-going BRTS physical model tests have demonstrated that the former should function satisfactorily in this respect and other tests are in progress to investigate the latter.

If the US type proves to perform as well as they Indian type then this would be preferable since construction control will be considerably simpler.

There are arguments for and against continuing the geotextile membrane under the toe/apron. In favour is the elimination of the risk that sand will migrate from behind the deformed protective layer and thereby initiate a collapse of the sloping layer above to a steeper slope. If this were to occur it could seriously increase the likelihood of geotechnical slope failure. In practice there is some evidence that such migration is not a serious problem, perhaps because after deformation has occurred the flow velocities in the immediate vicinity of the deformed layer are relatively low and the thickness of the layer is sufficient to prevent further material movement. The counter arguments are that firstly the relatively low friction of the membrane may result in an uneven distribution of blocks during deformation and secondly that there are practical difficulties over placing the membrane in such a way that it can follow the deformation process.

129

There are also arguments for and against the use of dumped rock in the toe/apron in place of blocks. In favour is the better protection that graded rock can offer in the absence of a filter membrane. The counter argument is that segregation may occur during the deformation process resulting in uneven displacement of the material. Since there is no simple way of testing and comparing these two factors in practice, it would seem that the choice will again depend on relative cost and the weighting given to the employment of unskilled labour.

5. TERMINATIONS

The results of the BRTS physical modelling have shown that the upstream terminations, which are perhaps the most crucial element of the whole structural system, are likely to be exposed to the most severe hydraulic conditions. A second consideration is that their geometry does not lend itself to the use of mats for the sloping faces and special techniques would have to be developed for the controlled placing of blocks.

The implication of the first consideration is that in general the design approach should be more conservative for these elements. It may prove desirable for example to specify the use of a larger block on those parts of the sloping faces that are most exposed.

In other respects the arguments presented for the other elements are applicable to the respective parts of the terminations.

6. CONCLUSIONS

The introduction of modern heavy duty geotextiles has changed the emphasis away from the armouring layer as the principle component of the protective system to the geotextile membrane. The primary function of the armouring layer is to maintain the membrane in close contact with the underlying soil and to protect the membrane material from exposure to ultra-violet light and mechanical damage.

The current BRTS standard revetment design is based on well tried and proven Indian Practice, which was developed before the introduction of modern geotextiles. Seen in this light, the current design for the slope protection above LWL is considered to be unnecessarily conservative. One layer of blocks of the appropriate size would fulfill the functional requirements, given suitable placement control and provision for ensuring that the membrane cannot become exposed to ultra-violet light.

For the slope protection below LWL the current design of the protective system is seen as appropriate. The nominal two layer thickness specification for the armour zone provides a satisfactory basis for ensuring that coverage by at least one layer will be achieved in practice over the whole face, which is the absolute minimum functional requirement. The specification covering the placing of the material should be consistent with this principle.

Physical model tests on different toe/apron configurations are in progress. In general a more compact toe is favoured with regard to ease of control during construction, which should result both in better performance and lower placement cost. The role and performance of a geotextile membrane under the toe is unclear and will remain so until sufficient data on prototype performance becomes available. In general, at this stage, it would seem prudent to provide the membrane in situations where this does not entail additional construction operations specifically for this purpose or other excessive additional costs.

The upstream terminations will be exposed to the most severe hydraulic conditions and therefore a rather more conservative approach is appropriate. Consideration may be given to the specification of larger armouring units in those parts of the sloping face that are most critical than for the remainder of the revetment.

In principle the use of rock as an alternative to concrete blocks is considered to be technically equivalent in terms of performance. Well graded rock will have some advantages as slope armouring if correctly placed (i.e provided segregation can be avoided) but there is no reliable evidence as to its relative performance in the toe/apron during deformation. Choice between these two materials will be influenced by the cost of materials, the contractor's access to suitable plant and the weighting placed on the use of unskilled labour during construction.

The use of other systems, such as bitumen grouted rock, for the slope above LWL may also be appropriate but there is no first-hand experience of its performance under the prevailing conditions. The pilot studies under FAP-21/22 have been specifically set up to test such alternatives. Until these results are available it is considered prudent to follow well tried techniques.

In the longer term the use of flexible mattresses consisting of concrete blocks bonded onto the geotextile membrane are seen as the most cost-effective system provided that a sufficiently large scale and sustained implementation programme can be ensured in order to justify the very large initial investment in specialised plant, and subsequent high recurrent costs, that this entails.

Dr Fleming will provide examples of the use of single sized concrete blocks and other techniques and systems from Halcrow experience.

CAFleming
16/1/92

APPENDIX D

DESIGN NOTE ON REVETMENTS FOR PRIORITY WORKS

132

BANGLADESH WATER DEVELOPMENT BOARD

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

DESIGN NOTE ON REVETMENTS FOR PRIORITY WORKS

APRIL 1992

SIR WILLIAM HALCROW & PARTNERS LTD.
in association with

Danish Hydraulic Institute
Engineering & Planning Consultants Ltd.
Design Innovations Group

BANGLADESH WATER DEVELOPMENT BOARD
RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

DESIGN NOTE ON REVETMENTS FOR PRIORITY WORKS

CONTENTS

134

TABLES

FIGURES

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER
A NOTE ON THE DESIGN OF REVETMENTS FOR PRIORITY WORKS

1. INTRODUCTION

This note is intended to expand and update the design criteria and methodology for the Priority Works which have been set out in Appendices B1 and B2 to Annex 6 Part 1 of the BRTS First Interim Report and in 'Report on Priority Works', October 1991.

In particular, the different parameters relating to revetment design are finalized.

2. TYPE OF PROTECTION WORKS

The strategy for the design of the Priority Works is to use materials which have been tried and tested previously on the Brahmaputra river and other major rivers in Bangladesh viz. individual concrete blocks hand placed over a filter layer on the bank slope above LWL and dumped below LWL to form the lower revetment and falling apron. As an alternative to concrete blocks, quarry rock can be used on the slope below LWL and for the apron, where boulders can also be used.

It has been demonstrated by model tests and cost and technical calculations that revetments will be the most cost effective bank protection measure to be adopted at each of the priority sites, except where the existing Kalitola Groyne will be rehabilitated at Sariakandi. At Sirajganj, it is planned that the Ranigram Groyne be incorporated into the new revetment at its upstream termination (see Figure 2.1).

3. FORM OF CONSTRUCTION

The same basic system will be employed at each site: a layer or layers or concrete cubes overlying a filter formed by dumping/placing a layer of bricks on geotextile filter fabric. Above LWL the bricks will be crushed to 50 mm size and below whole bricks will be placed on geotextile/bamboo fascine mattresses.

The existing bank slopes are very variable, ranging from steeper than 1V:2H to flatter than 1V:10H. As a precaution against localized percolation or other pore pressure induced instability, any slopes steeper than 1V:3.5H will be treated either by trimming back the upper part or by dumping material in jute or geotextile bags to produce the required gradient of 1:3.5.

Above low water level plus 2 m one layer of cubes will be laid side by side on the filter. This is accordance with sections 2 and 6 of the review of BRTS revetment design carried out by Dr C A Fleming of Sir

\36

William Halcrow and Partners Ltd. and submitted on 20 January 1992 (ref. HBD/BRT/615/662). Below low water level the cubes will be dumped from barges; the number of blocks corresponds to two layers random placed including 40% porosity.

The revetment will be taken down to the "Apron setting level" and a falling apron will be installed at the toe of the revetment, formed by dumping concrete cubes (or quarry rocks or boulders) of the same size as those used for the revetment. The above construction details are show in Figure 3.1.

At its upstream limit, the revetment will be returned into the bank to provide the required level of security against outflanking. At Sirajganj, the revetment upstream termination will join the existing Ranigram Groyne. At its downstream limit the revetment will be returned into the bank.

4. REVETMENT SLOPES AND RIVER BANK PROFILES

4.1 Revetment Slopes

Final geotechnical assessments and calculations including conditions during a 1:50 year earthquake show that a flat slope of 1:3.5 is required for long term stability.

4.2 River Bank Profiles

4.2.1 Above LWL

Excavation and/or placing and compacting of fill will be carried out as necessary.

4.2.2 Below LWL

Where the existing bank line is below the design profile, it will be built up using sand-filled jute or geotextile bags. These are preferred to khoa filled gunny bags for the following reasons:

- larger bags are more resistance to displacement by current velocities
- geotextile bags are more elastic and stronger than gunny bags. During dumping operations, the elongation property of the material rather than the tensile strength is more important.
- fill tends to be washed out during dumping or when the velocity is greater than 1.0 to 1.5 m/s.
- sand is readily available (by dredging) and cheaper.

If the existing bank line is above the design profile, it may be either excavated and trimmed or dredged.

5. REVETMENTS

5.1 Concrete Blocks

Concrete blocks are produced using either khoa (Jhama brick chips) or stone aggregate. The latter produces a concrete of higher specific gravity (2.24) and hence a slightly smaller block is needed to provide the same resistance to displacement by current or wave action.

Concrete cubic blocks (khoa aggregate) have been assumed for the Priority Works. It is considered that if high concrete quality control is maintained, the blocks will have sufficient abrasion resistance.

Preliminary estimates show that the number of bricks required for aggregate production is more than 150 million. Provision of such quantities from local sources may not be acceptable from an environmental viewpoint. Provisions will be made therefore in the Tender Documents for tenderers to submit prices for concrete blocks with stone aggregate to establish whether such blocks are cheaper when produced on a major scale.

When placing concrete blocks above LWL it is necessary to provide some degree of porosity for pressure relief of seepage forces in the underlying filter and soils. By examining existing revetments at Sirajganj, it has been concluded that there will always be sufficient gaps between the blocks to achieve the required porosity. Further provision could be made, however, by spacing the blocks, chamfering the corners or casting the blocks with a central hole.

For blocks which are dumped below LWL, a sufficient degree of porosity will be achieved to obviate the need for special pressure relief measures.

Conventional concrete cubic blocks are consequently specified. It is important to note that at Sirajganj more than 100 000 blocks are expected to be recovered from existing structures and reused.

5.2 Revetment Material Size

The material comprising the outer protective layer of the revetment shall be brick (or stone) aggregate concrete cubes. The size shall be determined by Eq. 1 and 2, whichever of the following two criteria, flow and wave resistance requires the larger cube size.

The BRTS design has been based upon S_s = 1.98 (from Chittagong O & M Circle) for brick aggregate concrete. The usual value taken by BWDB for design is S_s = 2.08, and on that basis there is an inherent factor of safety in the BRTS design. Shallow water condition (h/D = 5) is assumed as the worst case. The angle of internal friction (θ) for concrete blocks is taken to be 40°.

138

The critical parameters determining block dimensions are the near-bank velocity, significant wave height (H_s) and wave period (T). Values of near-bank velocity for the 1:100 year flood at each site have been derived from the hydraulic studies.

The analysis of wave conditions, shown in the Report on Priority Sites of October 1991, concluded that a design wave of H_s = 1.0 m and T_s = 3.0 s is to be used.

(a) To resist near bank flow velocity a cube size D is required:

$$D - \frac{0.7\ v^2}{2\ (S_s\ -\ 1)\ g} \cdot \frac{2}{[\log\ (6\frac{h}{D})]^2} \cdot \frac{1}{(1\ -\ (\frac{\sin\phi}{\sin\theta})^2)^{\frac{1}{2}}} \quad \ldots\ldots (1)$$

Where:

D = dimension of cube (m)
V = maximum velocity close to bank (m/s)
S_s= specific gravity of cube material
= 1.98 for brick concrete
g = gravitational acceleration
= 9.81 m/s²
h = depth of water (m)
ϕ = bank slope (degree)
θ = angle of internal friction of slope (degree)
= 40°

b) To resist wave action.

Above low water level, the revetment will comprise hand laid blocks, which will, if properly constructed, present a smooth surface to the flow and will not be subjected to large drag forces by the current. The physical model tests have further shown a reduction in flow velocity at the upper part of the slope. Wave action is consequently the critical factor.

The size of revetment armour required to resist wave forces has been determined from the stability equation developed by Pilarczyk.

$$D - \frac{H_s}{S_s\ -\ 1} \cdot \frac{1}{\beta} \cdot \frac{E^{\frac{1}{2}}}{\cos\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where:

D = cube dimension (m)
H_s = significant wave height (m) = 1.0 m
S_s = specific gravity of revetment material
= 1.98
β = strength coefficient
= 3 for cubes
= 2 for randomly dumped cubes

E = wave breaking parameter

$= 1.25 \frac{T}{H_s^{1/2}} \tan \phi$

T = wave period (s) = 3.0

ϕ = bank slope

This gives D = 0.55 m for ß = 2 and D = 0.37 m for ß = 3.

5.3 Final Determination of Block Sizes

Table 5.1 shows the block sizes (concrete cubes) corresponding to the design velocities found in the Note on "Design Velocities" using Equation (1).

Type of Structure	Amplification Factor	Design Velocity [m/s]	Calculated Block size (m)	Selected Size (m)
Revetment straight section	1.1	3.7	0.51	0.55
Revetment upstream termination	1.3	4.4	0.71	0.72
Head of * groyne	1.4	4.8	0.85	0.85

* including upstream termination at Ranigram Groyne, Sirajganj

Table 5.1: Type of Structure and Size of Concrete Cubes. (Probability for exceedance of 100 year design event is 1% in project life time (30 year)).

6. FILTERS

6.1 Above LWL

It is proposed that a non-woven geotextile filter is provided, that complies with the following:

- filtration efficiency (soil particle retention and permeability) (see the working paper on geotextile selection forwarded on 22 February 1992, ref. HBD/BRT/615/735).

140

- adequate tensile strength and elongation to resist damage during placement of concrete blocks.
- long-term resistance to degradation by UV-light
- high friction resistance to soil slopes

6.2 Below LWL

The only satisfactory way of providing on effective filter below LWL is to install a geotextile mattress. The geotextile should be non-woven and have the permeability, porosity, strength and elongation characteristics referred to in Section 6.1 above.

Installation of a geotextile mattress under water, although difficult, can be achieved with satisfactory results.

For a mattress to be laid under water, it requires:

- transverse and longitudinal stiffness (bamboo-frame)
- ballasting (whole bricks or crushed stone of similar size).

7. SCOUR DEPTHS

The studies of water depths and scour in the Brahmaputra have just been completed. The results will be presented in a separate report. It is concluded that a maximum water depth of 29 m in the 100 year design flood should be used for revetment design (straight sections). At the upstream termination more scour occurs and a water depth of 33 m is assumed in the design.

8. FALLING APRON (LAUNCHING APRON)

The existing lowest bed levels adjacent to the bank vary significantly over the reaches to be protected. Thus the calculated width of the falling apron (which is proportional to the difference between existing bed level and the lowest bed level that can occur) will be highly variable. To simplify construction, a limited number of standard widths (depending on the variation of the lowest bed levels along the particular reach) shall be adopted for each site. The additional material utilized in this way will not be significant and this will greatly facilitate placement control, with consequential overall cost savings.

The volume of material to be provided in the falling apron shall be determined by the following:

d = depth of scour below apron setting level.

D = size of cube or stone in the apron.

The volume is calculated assuming that:

(a) the face of the scour hole will be stable at a slope or not steeper than 1V:2H.

(b) The apron length shall be 1.5 x d and its thickness (based on 40% porosity) shall correspond to 3.5 layers of cubes randomly placed (Apron thickness: 3.5 x D) on straight sections and 4.5 layers randomly placed on the upstream termination. This detail is shown diagrammatically in Figure 8.1.

Theoretically, the thickness of armour on a scoured slope occurring below the setting level need be no greater than that above the setting level, i.e. 2D where D is the block size. Indeed model testing has indicated that a single layer of blocks after launching will prevent further scour. If d is the depth of scour below setting level, and the scoured slope develops to 1V:2H, the length of slope is $d\sqrt{5} = 2.24$ d. The width of apron is 1.5d, so the thickness of blocks on the apron to give 2D on the scoured slope is 2D x 2.24/1.5 = 3D. Applying a safely factor of 1.15 against adverse distribution, the thickness on the scoured slope becomes 2.3D and on the apron it becomes 3.5D, which is now proposed. This gives a ratio of thickness of apron to thickness on slope above setting level of 3.5/2 = 1.75, compared to a ratio of 1.9 proposed by S K Garg (Irrigation Engineering and Hydraulic Structures, Khanna Publishers, 1987).

9. UPSTREAM TERMINATION OF REVETMENT

At its upstream limit the revetment will be returned into the bank, as specified in the construction drawings. See proposed details on Ranigram Groyne Figure 3.1.

A falling apron will be formed at the foot of the slope. The length of the apron shall conform to the criteria set out in Section 8 above.

After construction the whole area within the return shall be backfilled to revetment level.

10. DOWNSTREAM TERMINATION OF REVETMENT

The design shall follow the same procedure as for the upstream termination, however the structure has a smaller extension as the downstream termination is less exposed than that upstream.

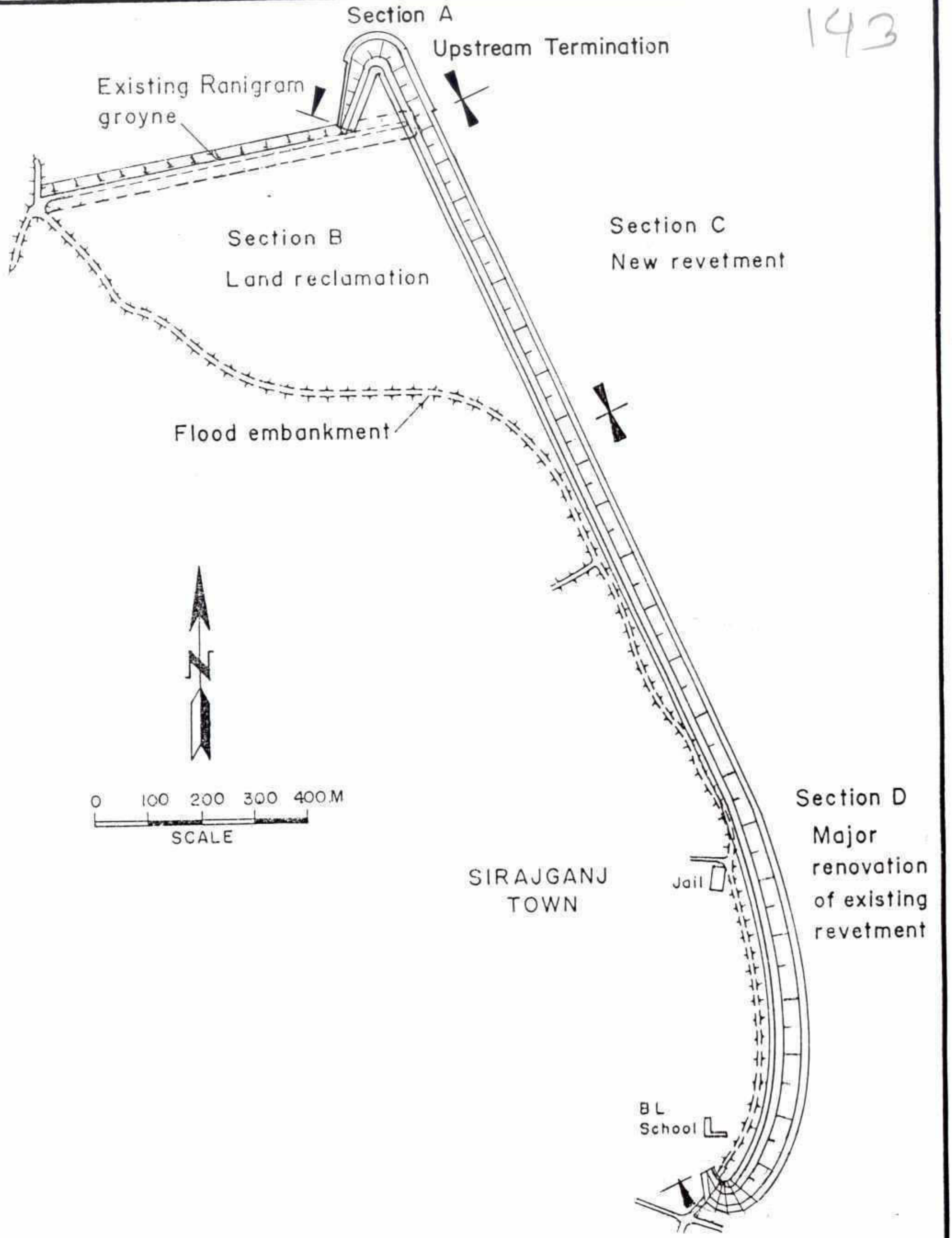
142

11. DESIGN WATER LEVEL

The levels given in Table 11.1 shall be adopted for the design of the priority works.

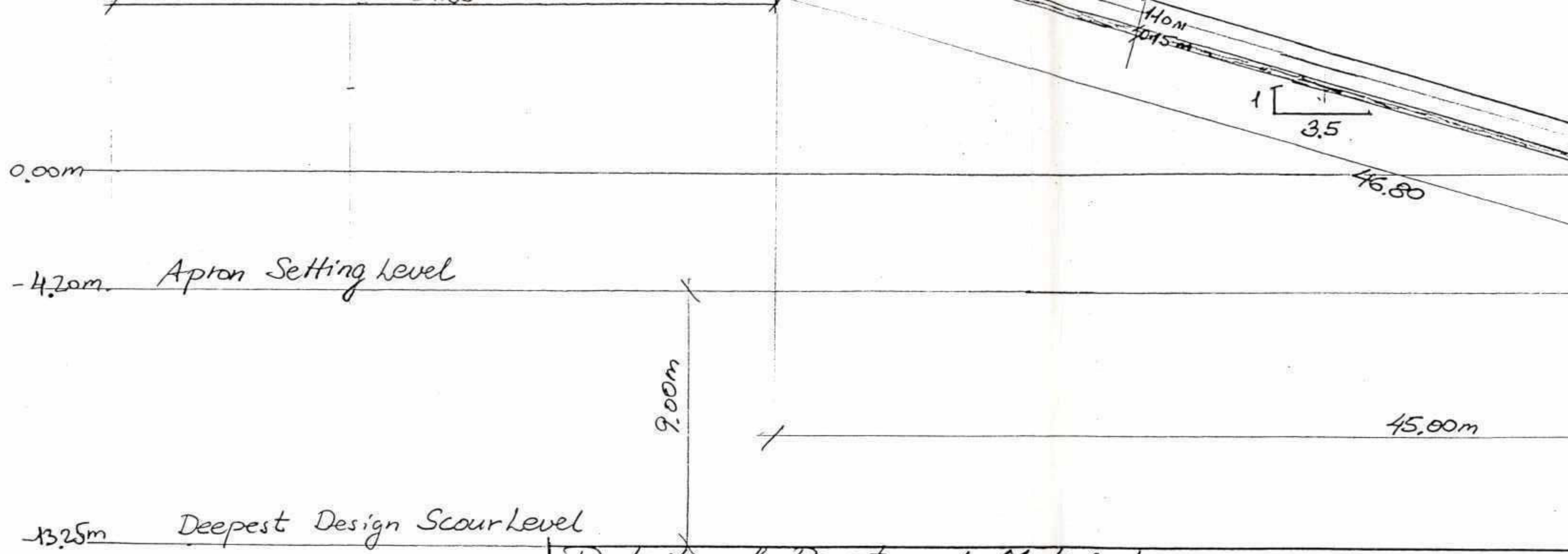
Table 11.1: Design Water Levels for the Priority Works

Location	Low water level (m PWD)	100 year flood level (m PWD)
Sirajganj	6.80	15.75
Mathurapara	11.18	19.45
Sariakandi	11.43	19.77

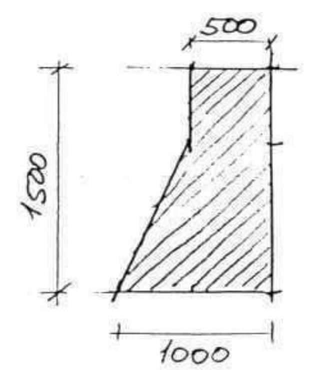


SCHEMATIC LAYOUT OF PROPOSED BANK STABILISATION AT SIRAJGANJ

Figure 2·1



Detail of Crest Block
scale 1:50 (mm)



<u>Details of Revetment Materials:</u>

<u>Upper-Slope</u>

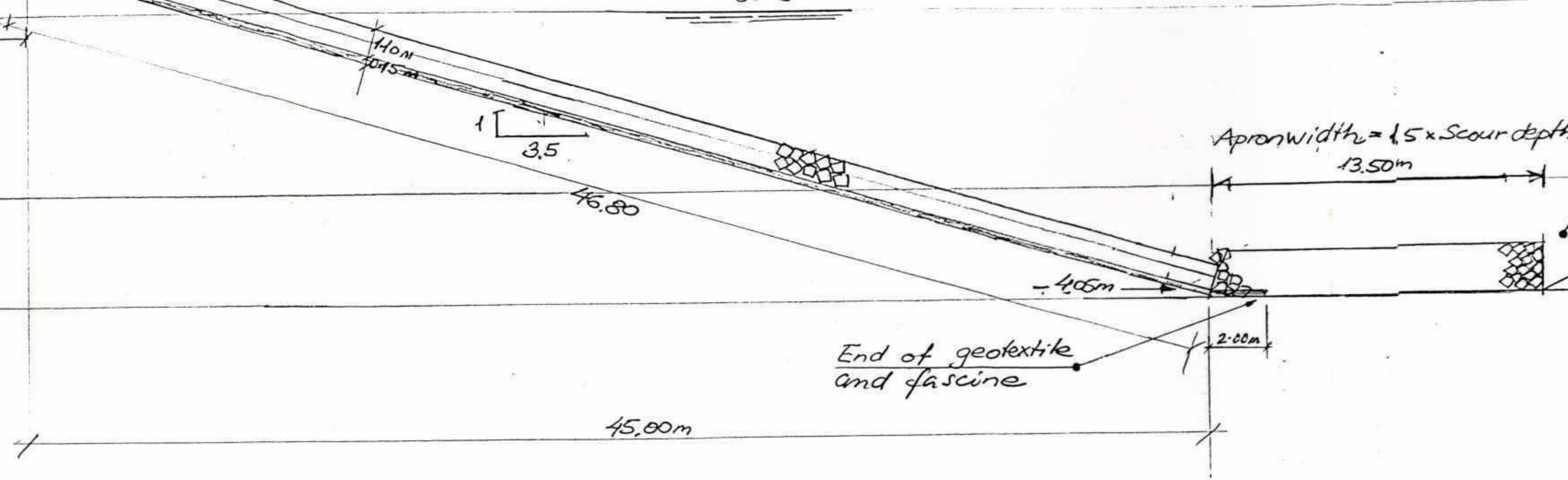
Layer density: 3.3 blocks/m^2
Total Number of Blocks: 25.84 × 3.3 ≈ 85 blocks/ Linear m. of reveti

<u>Lower Slope</u>

Layer Thickness: $r = n \cdot c \cdot V^{1/3} = 2 \cdot 1 \cdot 0.166^{1/3} = \underline{1.10m}$ (n = nu
Number of Blocks:
$N = n \cdot c \cdot (1 - P/100)\, V^{-2/3} = 2 \cdot 1 \cdot 0.60 \cdot 0.166^{-2/3} = \underline{4.0 \text{ blocks}/m^2}$ (p = p
Total Number of Blocks: 46,80 × 4.0 = 187 blocks/ Linear m. of reve

<u>Quarry Rock Alternative</u>
Size of rock: $D_{50} = 0.32m$ Assumed $\overline{W} = 85kg$ and $50kg \leq W$
Layer Thickness: $r = n \cdot c \cdot V^{1/3} = 2 \cdot 1 \cdot 0.033^{1/3} = \underline{0.64m}$
Number of Blocks:
$N = n \cdot c \cdot (1 - P/100)\, V^{-2/3} = 2 \cdot 1 \cdot 0.60 \cdot 0.033^{-2/3} = \underline{11.66 \text{ stones}/m^2}$ (Weig

<u>Apron:</u>
Concrete Blocks: Cross-sectional area = 13,50 × 1,93 = 26,00 m^2/m: Number
Quarry Stones or Boulders: ——— " ——— = 13,50 × 3,5 × 0,32 = 15,12 m^2/m: Number



f Revetment Materials:

e

: 3.3 blocks/m^2
r of Blocks: 25.84 x 3.3 ≈ 85 blocks/ Linear m. of revetment.

e

ness: $r = n \cdot c \cdot V^{1/3} = 2 \cdot 1 \cdot 0.166^{1/3} = \underline{1.10m}$ (n = number of layers, c = shape factor, V = volume

Blocks:
$P/100) V^{-2/3} = 2 \cdot 1 \cdot 0.60 \cdot 0.166^{-2/3} = \underline{4.0 \text{ blocks}/m^2}$ (p = porosity = 40%).

er of Blocks: 46.80 x 4.0 = 187 blocks/ Linear m. of revetment.

Alternative

ck: $D_{50} = 0.32m$ Assumed $\bar{W} = 85kg$ and $50kg \leq W \leq 150kg$. Density - 2.6 t/m^3, $\bar{V} = 0.033 m^3$

ness: $r = n \cdot c \cdot V^{1/3} = 2 \cdot 1 \cdot 0.033^{1/3} = \underline{0.64m}$

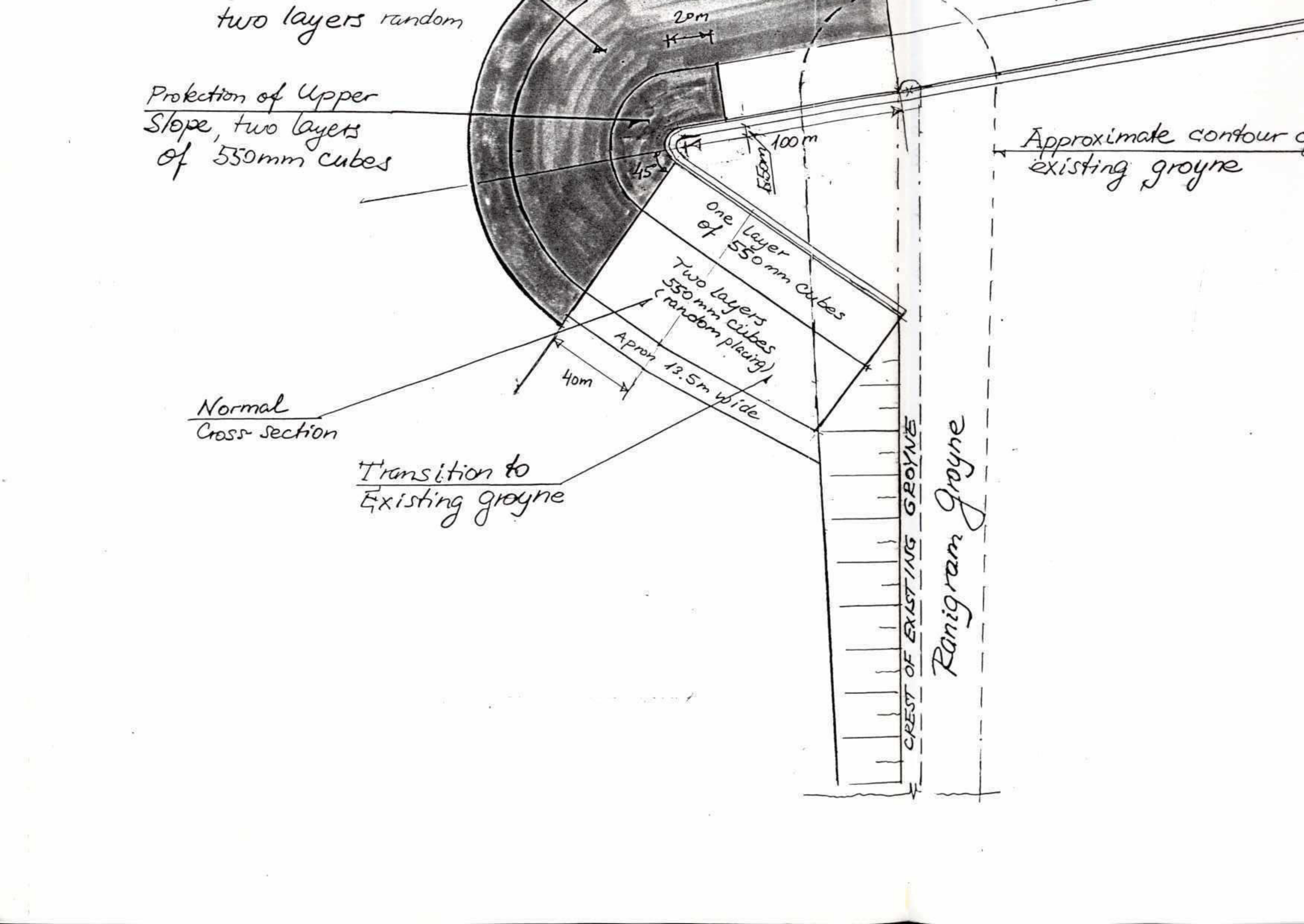
Blocks:
$P/100) V^{-2/3} = 2 \cdot 1 \cdot 0.60 \cdot 0.033^{-2/3} = \underline{11.66 \text{ stones}/m^2}$ (Weight of armour layer: 11.66 x 0.085t = $\underline{1.0 t/m^2}$)

ocks: Cross-sectional area = 13.50 x 1.93 = 26.00 m^2/m: Number of Blocks: 26.00 x 0.60 / 0.166 = $\underline{94 \text{ blocks}/m}$,
Boulders: —— " —— = 13.50 x 3.5 x 0.32 = 15.12 m^2/m: Number of Stones: 15.12 x 0.60 / 0.033 = $\underline{275 \text{ stones}/m}$

Design

Velocity

Water lev


two layers random
20m
Protection of Upper
Slope, two layers
of 550mm cubes
45°
100m
5.50m
One layer
of 550mm cubes
Two layers
550mm cubes
(random placing)
Apron 13.5m wide
40m
Normal
Cross-section
Transition to
Existing groyne
CREST OF EXISTING GROYNE
Ranigram Groyne
Approximate contour of
existing groyne

146

APPENDIX E

SELECTION OF GEOTEXTILE FOR BANK SLOPE PROTECTION

BRAHMAPUTRA RIVER TRAINING STUDY

SELECTION OF GEOTEXTILE FOR BANK SLOPE PROTECTION

1. INTRODUCTION

1.1 Objective of this Note

The purpose of this note is to set down the functional requirements for geotextile membranes, in the specific context of the short-term works identified and designed under the BRTS, to compare these with the known material properties and thereby to reach a conclusion as to the most appropriate specification.

1.2 Outline Description of the Works

The short-term works consist of bank stabilisation measures over discrete lengths of the right bank of the Brahmaputra (Jamuna) river designed to safeguard specific infrastructural investment, e.g Sirajganj, or to prevent the occurrence of high consequential agricultural and sociological disbenefits, e.g. Sariakandi/Mathurapara. The works will consist of the upgrading of existing stabilisation measures, consisting of both groynes and bank revetment, and the construction of new bank revetment. The cross-bars connecting the new bank revetment with the realigned BRE will also require some slope protection on the upstream face.

In all cases the function of the geotextile membrane is to prevent progressive deformation of the bank profile due to the migration of the fine sand and silt particles through the armour layer. Such deformation results in oversteepening of the bank and consequent slip failure of the bank material, resulting in discontinuity in the protective layer leading in turn to further failure. Migration may be due to wave and current forces or seepage flow gradients on the falling river stage. The membrane is this a key structural component of the systems; the primary function of the armour layer that overlays being to provide protection to the membrane and to keep it in intimate contact with the bank material.

1.3 Geotechnical Characteristics

The soils onto which the revetment/slope stabilisation system will be constructed are mainly very uniform fine sands with some bands containing silt and clay fractions. These soils are amongst the most difficult that can be encountered in terms of engineering properties.

From the results of grading analyses carried out at the RRI on 46 samples taken from the river bank and 184 samples collected from the geotechnical investigations, it is found that the two limits are represented by the lines shown on Figure 4. Referring to the PIANC Guidelines for the Design and Construction of Flexible Revetments Incorporating Geotextiles for Inland Waterways, it is found that the finer material, which is a silt clay, falls into Class A while the very uniformly graded but somewhat

148

coarser material, with a d_{50} of about 0.4mm is Class B. Lying in between these two extremes are a large proportion that fall into Class C. For the purposes of soil retention the filter effective opening size, O_{90}, is the critical consideration and in this case it is the Class A material that will determine the O_{90} required.

The second important classification identified by PIANC is the susceptibility of the soil to downslope migration. This problem is most apparent in silts, sandy silts and fine sands. The minor cohesive properties of these types of soils tend to encourage high mobility of small single grains at relatively low hydraulic gradients.

The PIANC guidelines for recognising susceptible soils are firstly that a proportion of the particles will be smaller than 0.06, which is satisfied by almost all the Brahmaputra soils. Secondly that at least one of the following criteria shall be satisfied:

(a) the coefficient of uniformity $C_u = \dfrac{d_{60}}{d_{10}} < 15$

(b) 50% or more of the particles will lie in the range $0.02\text{mm} < d < 0.1\text{mm}$

(c) the plasticity index $PI < 0.15$

Since the PI of the majority of the Brahmaputra soils is effectively zero, this criterion alone will place the soils in the susceptible range. In fact almost all soils also satisfy criterion (b) and a large proportion satisfy criterion (a). There can be little doubt that the soils have to be considered as susceptible to downslope migration.

The treatment for such conditions is discussed below.

2. KEY FUNCTIONAL CONSIDERATIONS

2.1 Design Concept

As described in Section 1, the function of the geotextile membrane is to prevent migration of the fine sand and silt particles that would otherwise result in over-steepening of the slope and consequent geotechnical failure. Migration may be due to wave and current forces or seepage flow gradients on the falling river stage and mat take place down the slopę or through the armour layer. The membrane is this a key structural component of the systems; the primary function of the armour layer that overlays being to provide protection to the membrane and to keep it in intimate contact with the bank material.

2.2 Functional Requirements of the Geotextile

2.2.1 Filter Properties

The filter must have a porous structure in which the combination of fibres and spaces between them (pores) allows the soil particles to be retained in such a way that no blocking or clogging of the filter occurs. This structure is best achieved with a geotextile having the maximum proportion of pores per unit volume. This allows each soil particle size to find its appropriate place in the structure such that further particles are prevented from entering without seriously reducing the permeability of the geotextile. The pore ratio for different materials are shown in Table 1, from which it can be seen that needle-punched non-woven geotextiles offer the best solution in this respect. In addition to the requirement for high pore ratio, the thickness of the membrane is also an important consideration; the thicker the geotextile the better the distribution of soil particles and the less the likelihood of blocking or clogging.

The Effective Opening Size, O_{90}, is defined as being the grain size of a test "soil" corresponding to 90% retention by weight on a sample of the geotextile in a vibrating sieve apparatus. The German Standard and Swiss Standard are based on a standard sand in a wet apparatus whereas the British Standard uses graded glass beads in a dry apparatus. The American Standard is similar to the British. It is therefore important to be clear about the Standard that is being applied. The PIANC standard is based on the Swiss Standard SN640550.

Using the guidelines in the PIANC publication, the O_{90} for a Class A soil should satisfy three criteria

O_{90} $< d_{90}$, the lowest value of which is 0.075mm

$< 10.\ d_{50}$, the lowest value of which is 0.070mm

< 0.3mm.

Thus the filter fabric O_{90} should be less than or equal to 0.07mm

15

2.2.2 Downslope Migration

It was shown in Section 1.3 that the Brahmaputra bank soils fall clearly into the category of soils that are susceptible to downslope migration. There are three recognised ways of addressing this problem:

(a) by providing a thick stabilising layer of coarse fibres needle-punched to the undersurface of the geotextile. Experience has shown that the coarse fibres integrate with the soil surface, thus reducing downslope migration within this layer. Following the PIANC guidelines, the stabilising layer for these soils must satisfy the condition:

$$0.3 < O_{90} < 1.5\text{mm}$$

and the thickness of the layer, tgg, must be:

$$5 < tgg < 15\text{mm}$$

(b) Attenuation of the hydraulic gradient by providing a granular sublayer between the geotextile and the cover layer. This layer must be fine enough the provide adequate damping effect yet be coarse enough to be retained by the cover layer. The placement of such a layer underwater is considered to be impracticable and this approach is therefore not appropriate for the lower slope protection.

(c) Providing a sufficiently heavy coverlayer such that the uplift pressures caused by excess pore water pressure are resisted. It is considered that the combination of the proposed lattice with brick or other ballast and the concrete block armour layer will provide adequate loading provided a heavy geotextile of over 5 kg/m^2 is used.

2.2.3 Permeability

The PIANC Guideline recommends that the geotextile should be 50 times more permeable than the underlying soil. Applying Hazen's formula the permeability of the soil can be estimated as follows:

$$k_s = 0.0116.(d_{10})^2 \quad m/s$$

where d_{10} is measured in mm

The typical d_{10} of the Brahmaputra soils may for this purpose be taken as about 0.08mm which gives a k_s of 7.4×10^{-5}. This may be compared with the falling head permeability measurements conducted during the BRTS field investigations at depths of 15m and 30m, which produced a highest value of 4.6×10^{-5} m/s and a lowest value of 0.9×10^{-6} m/s. The mean of the 14 tests was 1.6×10^{-5} m/s and the Gumbel SD 1.2×10^{-5}. There is the possibility that the borehole falling head results may have been affected by some clogging of the stratum by traces of the drilling mud and som a safe assumption would be 7×10^{-5} m/s.

The permeability of the geotextile should therefore be not less than 3.5×10^{-3} m/s.

2.2.4 Thickness

It has been noted earlier in this note that the thicker the geotextile the less prone it will be to clogging. The PIANC guideline also states that experience has shown that for soils with susceptibility to downslope migration the minimum thickness should be 5mm. The German Federal Institute for Waterways (BAW) recommend a minimum thickness of 4.5mm for most soils but 6.0mm for what they classify as Soiltype 4, which it is understood are similar to the Brahmaputra soils.

A minimum thickness of 5mm is therefore indicated and this may be increased to 6mm if there is concern about the severity of the problem.

2.2.5 Tensile Strength

Provided that the material is sufficiently flexible to accommodate the deformation that will take place during placing, and this is considered in Section 2.2.7, then there is no specific strength requirement subsequent to laying. The determination of the strength requirement will therefore be by consideration of the stresses that will occur during laying.

Above water level the installation stresses will be relatively modest and a strip tensile strength of 12 kN/m will be sufficient. The strength requirement for laying below water level will depend on the method of laying adopted by the Contractor but it would be prudent to specify a minimum of 20 kN/m, with the provision that the Contractor could propose for the Engineer's approval a lower strength material on condition that he could satisfy the Engineer that his method of installation would induce excessive stresses. In any event the Contractor will have to demonstrate in his Method Statement that his method of installation is commensurate with the material properties. If the specified strength cannot be achieved with a single layer material, then the option is to use a combination of woven and non-woven needlepunched together.

2.2.6 Seam Strength

The seam strength should be related to the material strength. There is clearly no advantage in having an excessively strong seam but nor should it represent a weakness in the completed blanket. Particular attention has to be paid to loops attached to the geotextile for the purpose of fastening any form of stabilising or ballasting grid. Depending on the method of laying there are possibly advantages in using a composite geotextile for this latter application with the loops sewn to the non-woven element.

Experience and tests show that with non-wovens usually not more than 75% of the geotextile strength can be transferred through the seam. In general the seam strength may be specified as being equal to not less than 90% of the tensile strength of the geotextile.

152

The seam form may be the simple "prayer" type with the overlap between 50 and 100mm. Stitching should be double thread chainstich using a thread that will generate the required seam strength. Sample testing prior to commencing the main stitching is the only reliable means of confirming that the thread and stitching system is satisfactory.

2.2.7 Elongation

As noted in Section 2.2.5, the significance of elongation in the context of the BRTS short-term works is that the geotextile must be capable of deforming during installation to conform to the irregularities of the prepared profile. Where the profile contains existing blocks and other armouring material or is composed of jute or geotextile sand-filled bags the degree of irregularity may be considerable.

An elongation of at least 50% in the transverse and 80% in the longitudinal direction is required for this purpose and greater values would be no disadvantage unless this created handling difficulties.

2.2.8 Puncture Resistance

The robustness of the geotextile is a major consideration in relation to the portion of the revetment placed underwater where dumping of concrete blocks or graded rock for the armour layer will be options. If the geotextile is sufficiently robust in this respect it may be feasible to dump heavy angular material directly onto the geotextile; otherwise it will be necessary to provide a buffer layer of bricks, or possibly jute sand bags

Puncture resistance is a function of both fabric form and constituent material characteristics. In general the thicker and more flexible the fabric the more puncture resistant it will be but the type of construction is perhaps more important in this respect.

2.2.9 Abrasion Resistance

The design of the slope protection is such that the geotextile filter layer should never be exposed to direct abrasive attack from the suspended sediment in the river. Abrasion could occur if the fabric were able to move (flap) in contact with the armour layer. This would be most likely to occur in the event of the armour layer being placed directly onto the fabric.

If geotextile were to be specified under the falling apron then the action of apron deformation would cause abrasion as the armour material migrated. Once the system had become stable there could also be abrasion due to the flapping action mentioned earlier.

2.3 Construction Considerations

2.3.1 Above Water Level

The laying of the geotextile above water level has become a routine operation and one that offers no significant problems to an experienced contractor. Seaming can be down in situ using portable stitching equipment. The sometimes quoted advantages of heat welding in this respect are outweighed by the difficulties of ensuring that competent welds are achieved in practice.

2.3.2 Below Water Level

Laying geotextile below water level presents the following principal problems:

(a) Positioning the fabric, free of folds in its correct location with the proper lap;

(b) keeping the fabric in position until the ballast/armour layer has placed;

(c) maintaining the lap zone clear of material and the fabric stable until the overlaying sheet is in position;

(d) ensuring that the lap is free of material and the two sheets are free of folds prior to placing the ballast/armour layer.

(e) ensuring that the fabric does not become punctured during the process of placing the armour layer.

In the case of the Brahmaputra, this will have to be undertaken in turbid water with minimal visibility and localised flow velocities of up 1.0m/s during the construction window period. The main experience of carrying out such operations under similar conditions has been along the northern European coastline and in the USA. In both situations, the large scale of the works has made it worthwhile for contractors and government agencies to invest in sophisticated purpose built equipment. There are however examples of the use of less capital intensive techniques and it is from these that useful guidance may be drawn for application to the Brahmaputra short-term works, whose scale does not warrant the mobilisation of the larger and more sophisticated range of specialist equipment (unless a firm committment to an extended programme of such works could be given).

It is apparent from the problems summarised above that the primary requirement is for a fabric based system that has the appropriate combination of rigidity to ensure satisfaction of (a) and (b) while having adequate flexibility to facilitate handling. At the same time, the complete system has to be sufficiently dense to remain in close contact with the base material, in flow velocities of up to 1.0m/s, while ballasting/armouring is in progress.

154

The lap zone presents particular difficulties. The system must remain stable for longer, while the lap is being formed, and yet there must be the minimum interference with the proper functioning of the lap.

Probably the two main contenders for laying the geotextile system under water are:

1. the pull-out and sink method (Figure 1) and

2. the unroll underwater method (Figure 2).

The latter has the attraction of positive accurate positioning but in practical terms is dependent on a relatively lightweight and flexible geotextile system if very heavy equipment is to be avoided. The former is a more rugged approach, requiring little in the way of sophisticated equipment, that is suitable for the heavier and less flexible systems, such as fascine mattresses. A third approach is:

3. the lowering of large semiflexible mats into position using heavy pontoon mounted cranes (Figure 3).

This third method has the advantage of reasonably accurate placing, though not as good as (2), and elimination of any problems relating to folds in the fabric and displacement during ballast/armour placing. The disadvantage is that there is a practical limit to the mat size and therefore more joint problems.

The lapping difficulty is common to all three methods and is best minimised by the simple expedient of maximising the sheet/mat size.

2.3.3 Lap Widths

The particular difficulties associated with forming laps under water have been outlined in the previous Section. The lap width requirement is determined by the minimum overlap that will ensure that the membrane effectively acts as a continuous sheet. To this must be added an allowance for the positioning errors inherent, in the specific installation method and a further safety provision to allow for some lack of uniformity in the lap, including material folds and material inclusion between the layers. If the stiffening lattice interferes with the achievement of intimate contact between the lap layers then some further allowance must be made for this.

The geotextile manufacturer should advise on the lap width that he recommends for his material but it should not be less than:

at time of placing	Min lap width (m)
depth of water not exceeding 2m:	1.00
depth greater than 2m but less than 10m:	1.50
depth greater than 10m:	2.00

Note that all joints made above water level should be sewn in situ, except for those on slopes of embankments on the flood plain where an overlap of 500mm is permissible.

2.3.4 Resistance to UV Radiation

The sensitivity of certain geotextiles to degradation following exposure to ultraviolet (UV) radiation is well known. This subject has been addressed under the general heading of construction considerations because once installed in accordance with the design the geotextile should never become exposed to UV radiation.

The most likely times at which the geotextile may become accidently exposed to UV light are during transport and handling. If site stores are constructed inaccordance with the Specification this should not be a source of accidental exposure. During installation exposure is inevitable and must be controlled to keep it within acceptable limits. The intensity and duration of UV radiation varies considerably during the year but it would be too complicated to allow for this in the Specification and a practicable maximum exposure duration has to be adopted that allows the Contractor the maximum flexibility for his working method.

Tests carried out in Australia have shown that Polyester and treated Polypropylene geotextiles suffer little loss of strength when exposed to intense UV radiation for a continuous period of 4 weeks, the equivalent of 6 months of exposure to extreme sunlight conditions prevailing during summer in Queensland, at a Latitude of 23^0. This is in marked contrast to susceptible materials.

There would seem no reason therefore to impose stringent conditions provided that UV resistant material is specified. A requirement that the material should be covered within two weeks of first exposure is conservative and reasonable.

2.3.5 Material Storing and Handling

It has been noted that polyester can degrade under certain combinations of moisture and heat as can arise from storing the film-wrapped material outdoors. If the wrapping is damaged and moisture enters, the high temperatures within the bundle can result in degradation of the fibres. This should not be viewed as a serious problem but one that must be addressed in the Specification and paid due attention during construction supervision.

3. COMPARISON OF GEOTEXTILE MATERIAL PROPERTIES

3.1 Woven and non-woven

In this context, the only advantage that woven geotextiles have over non-woven is their tensile strength. It has been seen that this characteristic may have a role to play with regard to the installing of the filter layer in deep water, in which case the woven fabric would be used in combination with non-woven. In all other respects non-woven geotextiles are far superior as filter materials.

3.2 Continuous and Staple Fibre

The two main groups of non-woven textile are:

continuous fibre non-wovens, in which the fibres are continuously extruded and spun;

staple fibre non-wovens, which consist of short, cut fibres with a fibre length of 5 to 20 cm.

Both have their advantages from the point of view of quality control. The exponents of continuous fibres claim that there is less chance of inferior or incorrect fibre material being incorporated in the fabric. Conversely the staple fibre manufacturers claim that quality control of the material is easier because it can be monitored before use and, more important, they are not committed to any one fibre size as the extruders are, thus enabling them to tailor their fabric to suit particular conditions. The latter would seem to be the more significant.

Relevant performance advantages claimed for continuous fibres are:

greater strength for the same mass per unit area;

higher resistance to cyclical stress.

Equivalent advantages of staple fibre are:

better abrasion resistance;

higher elongation and thus better conformance to base profile.

The last of these factors is probably the most relevant in respect of the Brahmaputra short-term works, as noted earlier in this paper.

3.3 Mechanical, Heat and Chemical Bonding

Early non-woven geotextiles tended to be heat or chemically bonded. Under the former system the fibres are melted together at their crossing points and in the latter case they are bonded by the addition of a binding agent. The introduction of mechanical bonding (needle-punching) has seen a marked

157

improvement in most of the fabric properties, perhaps most notably in relation to overall robustness and puncture resistance.

3.4 Polyester and Polypropylene

The common raw materials for geotextiles are Polyamide (PA or Nylon), Polyethylene (PE), Polypropylene (PP) and Polyester (PES). Of these only the latter two are normally used for hydraulic filters.

In general the UV stability of PES is greater than that of PP however it is possible to chemically treat the PP such that it can equal if not better PES in this respect. One slight practical problem is that there is no simple way of inspecting the PP to ensure that it has been so treated. The simple test for PES is that a small sample will sink in a glass of water whereas the PP will float.

The most significant characteristic of PES is however its specific gravity of 1.38 compared with that of 0.91 for PP. This means that when laid under water the natural tendency for PP is to float whereas PES will tend to stay in place. This can be a very important consideration when making the underwater laps.

158

4. CONCLUSIONS

4.1 Common Properties

The soils involved in all parts of the works are broadly the same and so the specification of the O_{90} pore size will be common to all situations. Similarly it is reasonable to specify a common geotextile permeability. Other characteristics may be selected to suit the particular conditions.

4.1 New Revetment Upper Slope

The principle design consideration for this zone is that the filter system should resist downslope soil migration. With sufficient superimposed loading this can be achieved with a simple non-woven geotextile of appropriate weight and thickness. With the original design consisting of an armour layer of two layers of hand-placed concrete cubes, a non-woven needle punched fabric 5mm thick and with tensile strength of not less than 10 kN/m would be suitable. There are some marginal advantages in using staple fibre textile in the particular situations where profile forming by sand-bag is required, otherwise either staple or continuous fibre may be specified. There would seem to be little to choose between PES or UV stabilised PP textile in this situation.

Reducing the superimposed loading by specifying only a single layer of concrete blocks would make the stability more marginal and the inclusion of a stabilising layer needle punched to the underside of the non-woven fabric would be advisable.

Were open stone asphalt to be used then the loading would certainly be insufficient and the stabilising layer would again be required. In this case the Specification for the main layer may also have to be modified.

4.2 Revetment Lower Slope

The conditions in this zone are far more arduous than in the upper slope zone. Amongst other considerations, the protective system will be permanently submerged in turbid water and so any failure will not become apparent until it has reached a fairly advanced stage.

The principal design considerations are that the filter system should resist downslope soil migration, it must be possible to place the textile accurately without creases and folds and to form laps successfully, and placing of the armour layer must not damage the filter.

With two layers of concrete blocks and the brick ballast layer, the superimposed loading should be sufficient to permit the use of a simple non-woven geotextile of appropriate weight and thickness to resist downslope migration. A non-woven needle punched fabric 5mm thick would be suitable for this function. Again, there are some marginal advantages in using staple fibre textile in the particular situations where profile forming by sand-bag is required, otherwise either staple or continuous fibre may be specified. Because of the particular advantage offered by

(5?)

the higher specific gravity, in this case PES is the preferred material, irrespective of method of construction.

It will be necessary to reinforce the simple non-woven textile in order to provide some rigidity during installing so as to avoid, at the very least, creasing and folding of the fabric before the ballast and armour layers can be placed. The original proposal was for the fabrication of a bamboo "fascine" grid following the principle of the willow fascine mattress widely used in northern Europe for this purpose. It has been suggested that the supply of a sufficient quantity of suitable bamboo may cause difficulties and alternatives are being considered. If some form of semi-flexible frame is adopted, whether of bamboo or other material, then it would be advisable to specify a composite geotextile consisting of a woven fabric needle-punched to the non-woven in order to provide greater tensile strength, particularly at the points of fixing of the frame to the geotextile. A minimum tensile strength of 20kN/m would be required for this purpose. A similar provision would be required if gabions were to be used for ballasting and spreading the geotextile.

An attractive alternative to the use of a conventional woven fabric would be to use Netlon Tensar Grid to provide both the reinforcing function of the grid and the greater tensile strength of the system, while retaining the flexibility of the non-woven geotextile. With this arrangement the bamboo fascine would not be required.

Consideration has also been given to the use of a "sand-mat" consisting of a layer of sand sandwiched between two layers of non-woven geotextile. This material has been used successfully in similar applications in large German navigation canals and the manufacturer states that it is sufficiently robust to permit the dumping of concrete blocks directly on to the mat. The advantage would be the elimination of the whole process of placing the ballast layer of bricks.

4.3 Falling Apron

The results of the BRTS physical model test series suggests that the performance of the falling apron during deformation may be less satisfactory with an underlying geotextile than without one. There would in fact seem to be a definite possibility that the geotextile will become exposed and may then be expected to flap around in the turbulent flow conditions, encouraging local scour and causing further displacement of the armour layer. This has to be set against the possible longer-term advantages, after the system has stabilised, if the filter layer were to deform smoothly and uniformaly under the armour layer.

If a geotextile system were to be provided then it would require the same properties as for the Lower Slope Protection but with an added emphasis on ability to deform in a direction transverse to the river flow, while minimising the tendency to flap. Both strength and abrasion resistance would also become more significant; one of which would favour the use of continuous fibre and the other a staple fibre fabric.

160

4.4 Geotextile Bags

In some situations the upgrading of existing slope protection, both bank revetment and groyne slopes, will require the placing of sand-filled textile bags both to establish a stable slope profile and to act as a filter. This is not an ideal filter arrangement but where there is existing armour it is a practicable means of providing a filter function without the necessity of providing an intervening "smoothing" layer of sandbags or the equivalent.

The geotextile for this purpose should be heavier duty than would be used for bulk void filling, where the function of the bag is simply to hold the sand in position until the stabilising layer is in position. The primary function of the geotextile bag, setting aside its void filling role, is to act as a damper of the fluctuating forces arising from wave and current action that tend to destabilise the soil particles underlying the existing armour, resulting in progressive collapse and ultimate failure. This is a considerably less arduous condition than imposed on the geotextile underlying new revetment. At the same time they must allow free movement of seepage flow during the falling river stage so that piezometric pressures in the bank can be relieved. Finally they must be robust and flexible enough to accommodate themselves to the irregularities presented by the existing armour.

A suitable material for this purpose that has been successfully in similar circumstances in northern Europe is a 800g/m^2 short staple non-woven fabric made of either PES or UV stabilised PP fibre. The short staple fabric is advantageous in this case because of its ability to deform around the existing armour. Nominal 0.75m^3 sacks, filled to not more than 80% capacity, have been found to be well suited, being easy to fill, transport and place. The size of an unfilled sack being about 1.15m x 2.30m.

For simple void filling or bulk core filling underwater where the geotextile is functioning primarily as a robust bag, although the georeinforcing properties should not be ignored, a lighter weight material would be appropriate. The selection being based on the method of placing and the distance that the bag will fall.

4.5 Flood plain embankment slopes

The physical conditions found on the slope of cross-bars and flood embankments constructed on the flood plain are only moderately different to those on the upper bank slope. The main differences being that the material will be selected to contain more silt/clay fraction for binding purposes, the wave and current induced effects are less severe and seepage forces will in general be lower. The downslope soil migration problem will accordingly be less serious but it cannot be totally ignored. In the absence of a heavy superimposed loading, the treatment would normally be the provision of a geotextile with a coarse fibre layer backing. Under these less severe conditions it would be sufficient to specify a 5mm thick non-woven fabric. Either staple or continuous fibre would be suitable and there would seem to be little to choose between PES or UV stabilised PP textile in this situation.

METHOD-1 : Mat laid on river

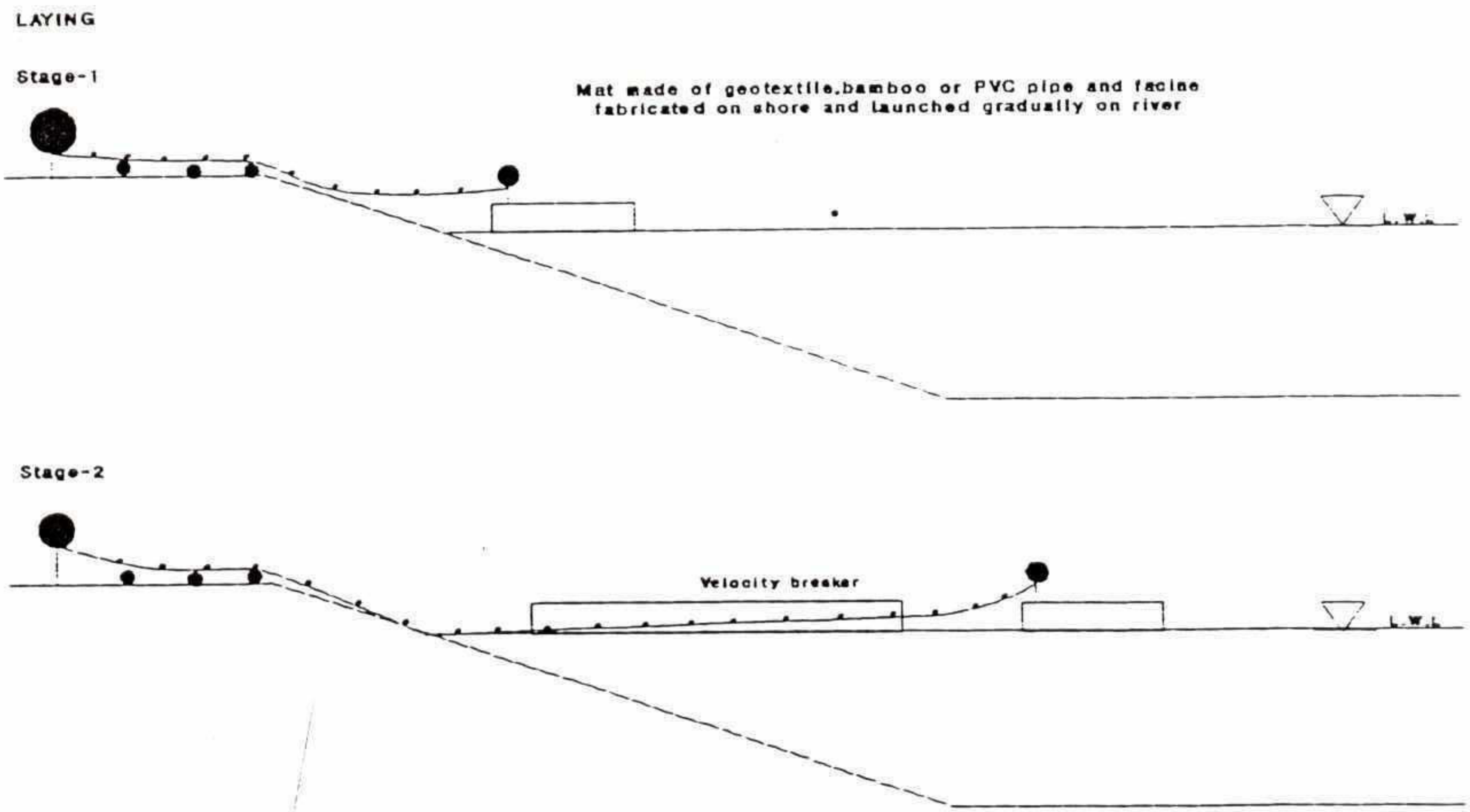


Figure : Ia

161

METHOD-1 : Mat laid on river

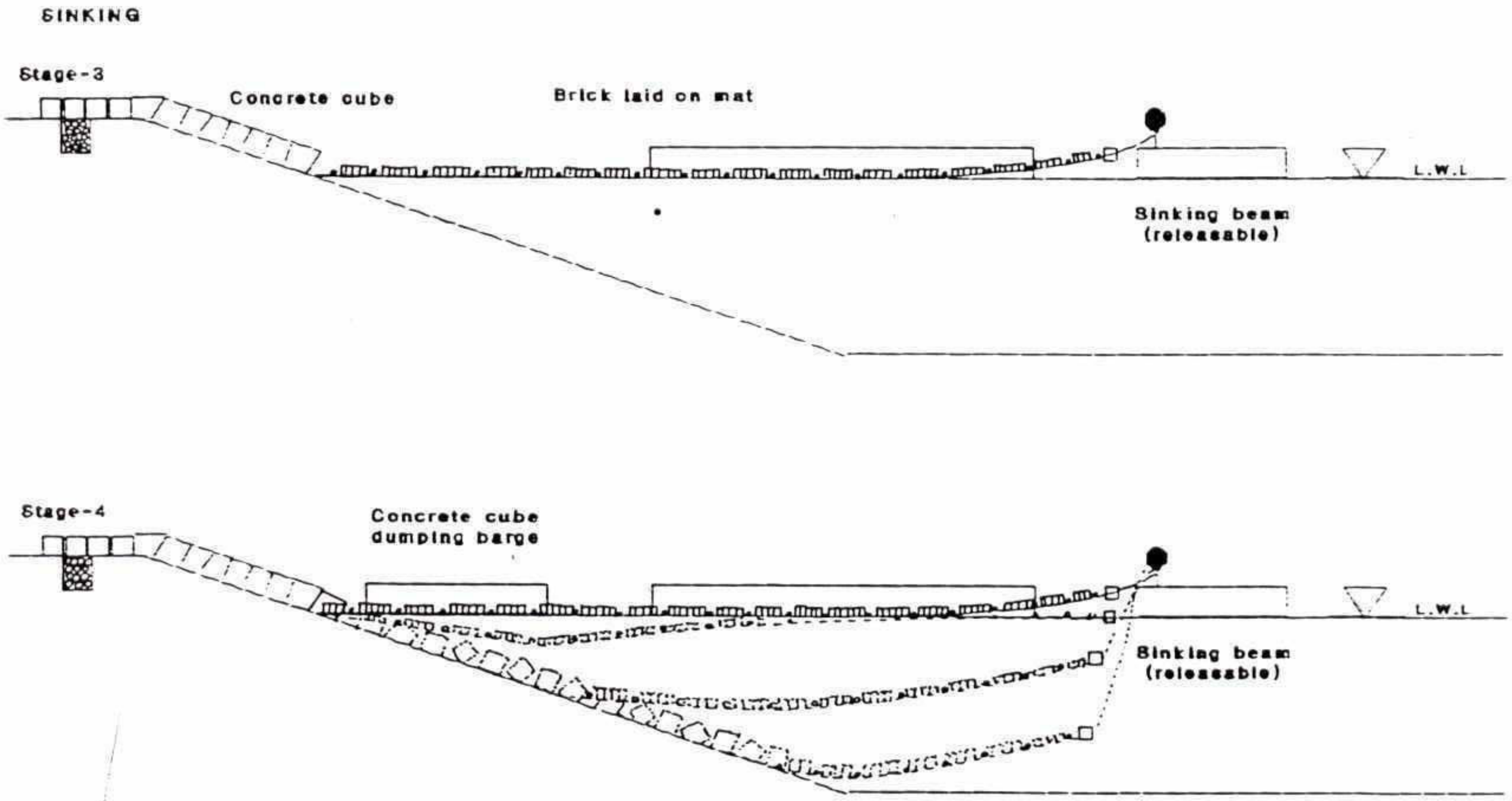


Figure: Ib

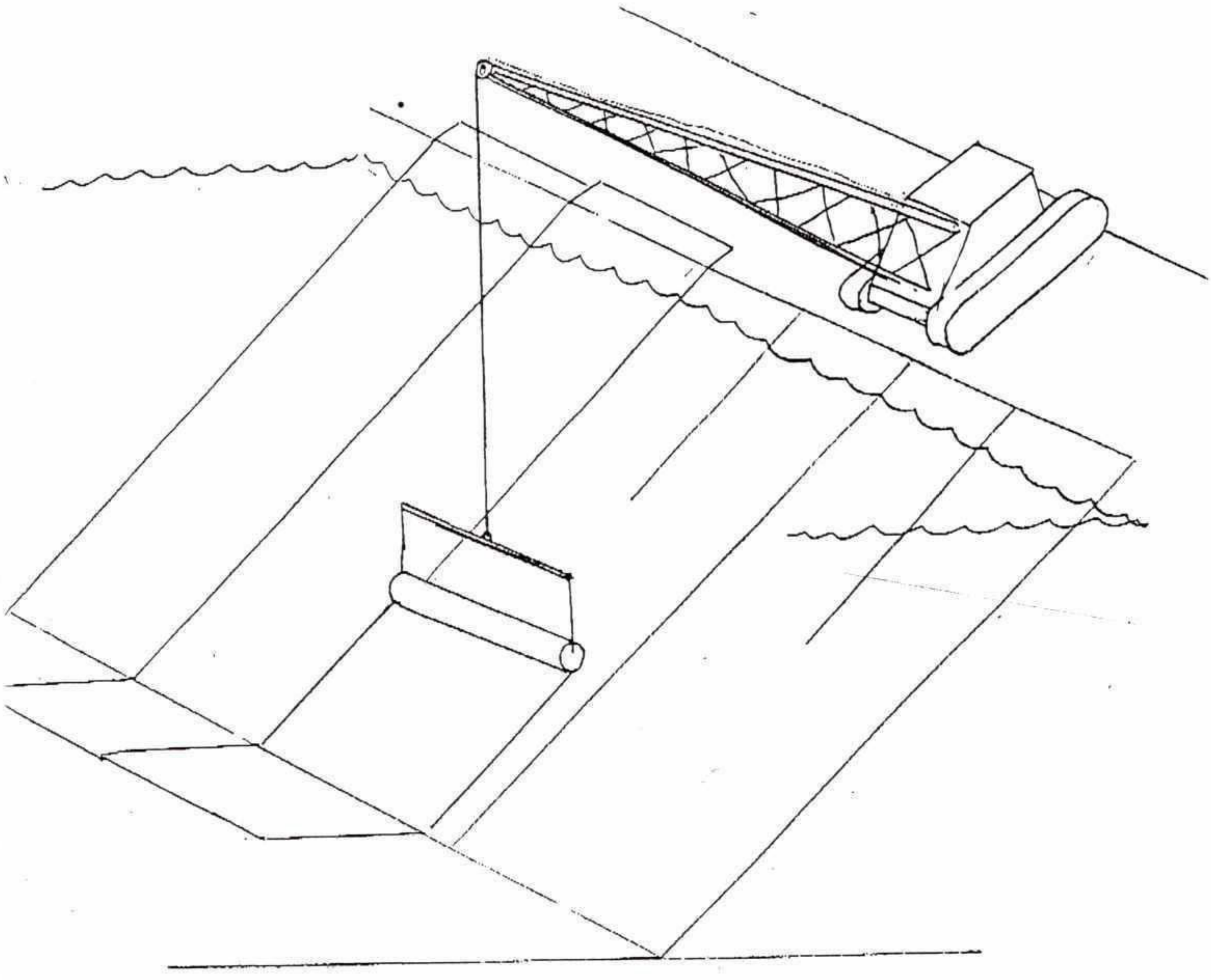
162

Figure : 2a

NAUE-FASERTECHNIK

163

METHOD - 2 : Unrolling Underwater

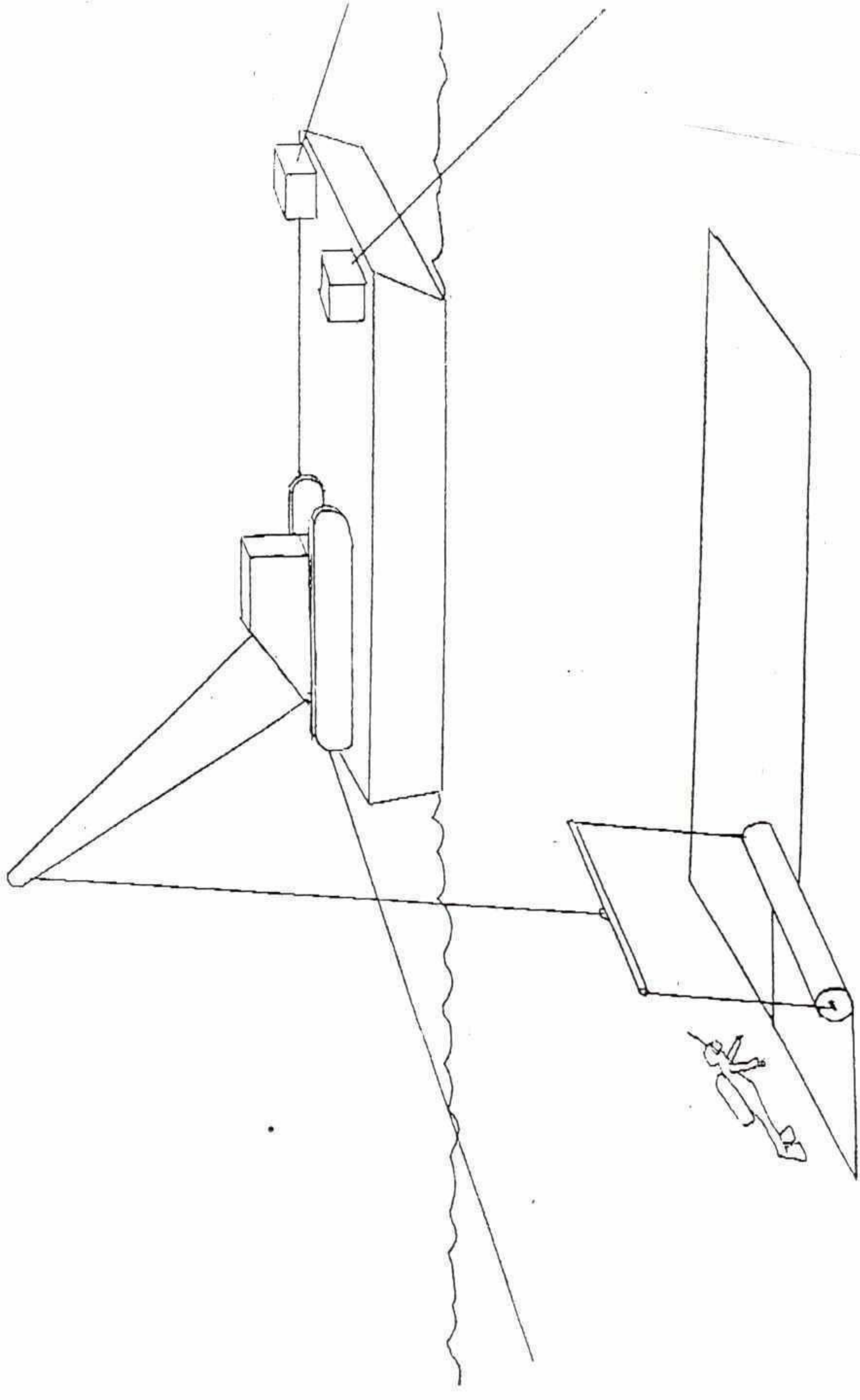


464

Figure: 2b

NAUE-FASERTECHNIK

METHOD-2: Unrolling Underwater

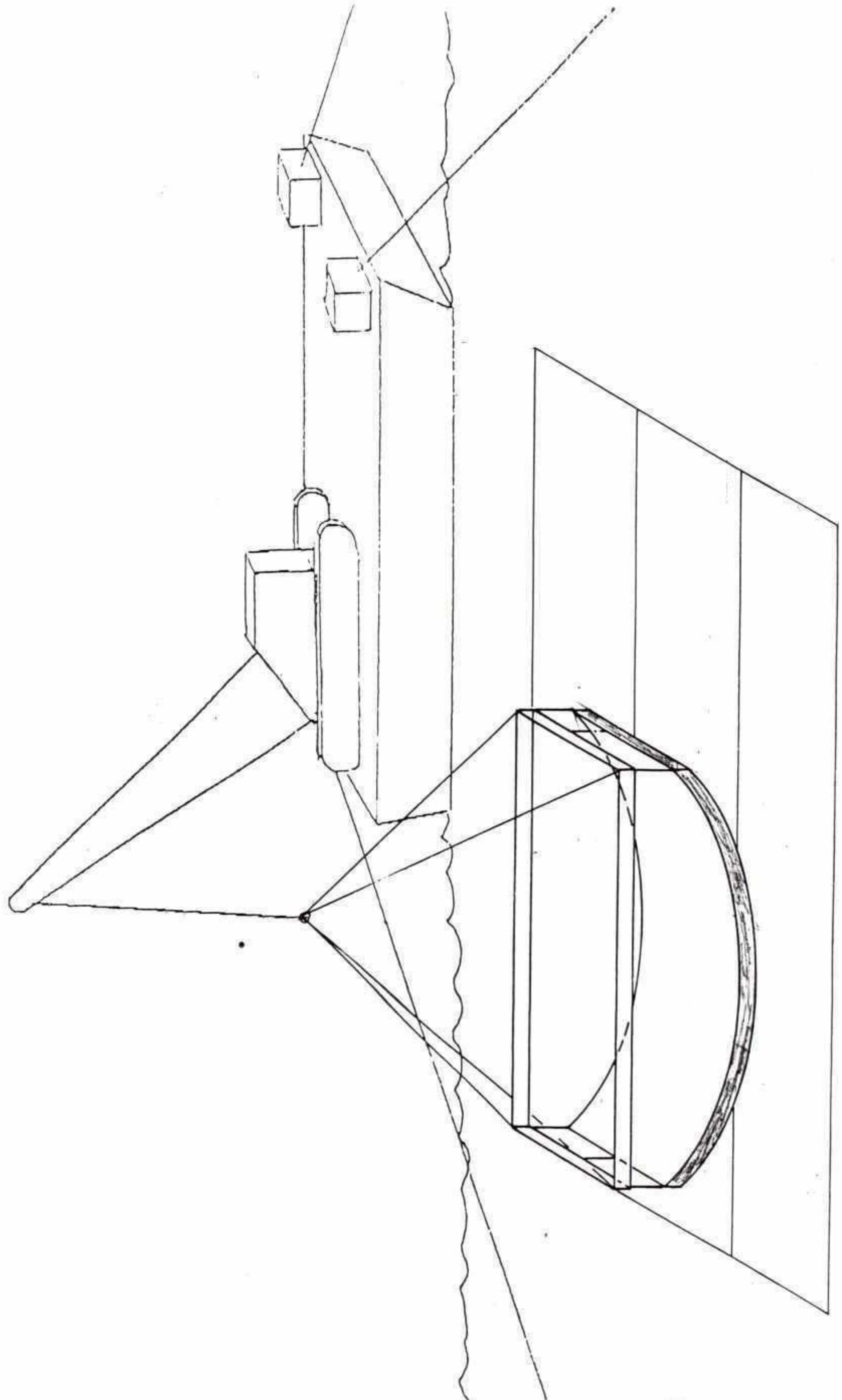


165

Figure: 3

NAUE-
FASERTECHNIK

METHOD - 3 : Laying Prefabricated Mats



Brahmaputra Right Bank
Soil Grading Limits

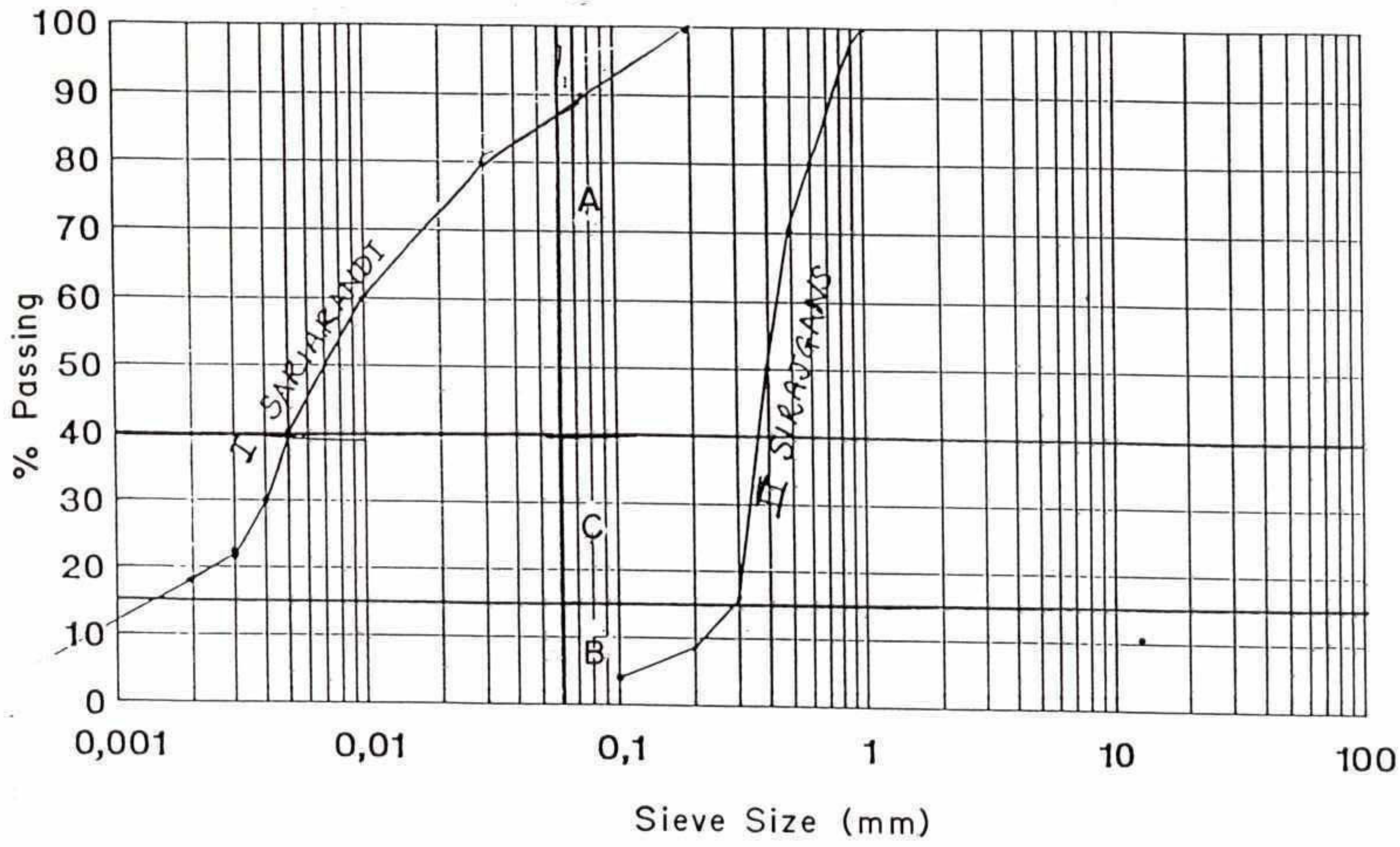


Figure: 4

166

167

APPENDIX

PIANC Guidelines for the Design and Construction of Flexible Revetments Incorporating Geotextiles for Inland Waterways

Appendix B: An Example of Design Procedure.

168

PERMANENT INTERNATIONAL ASSOCIATION OF NAVIGATION CONGRESSES

GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF FLEXIBLE REVETMENTS INCORPORATING GEOTEXTILES FOR INLAND WATERWAYS

Report of Working Group 4
of the
Permanent Technical Committee I

SUPPLEMENT TO BULLETIN N° 57 (1987)

General Secretariat of PIANC :
Résidence Palace, rue de la Loi 155, B. 9
1040 BRUSSELS (Belgium)

L69

APPENDIX B :
AN EXAMPLE OF DESIGN PROCEDURE

This design procedure is based on experience gained in West Germany [18] and has been used successfully for the following conditions,

Waterway	:	Class IV
Bank slope	:	1:3
Hydraulic loads	:	normal for a large navigation channel (see table 3.2)
Soil conditions	:	granular

The bank slope, type of soil and hydraulic load conditions combine to cause a situation where downslope migration of soil particles is often a serious problem and so, in effect represents a 'worst case' condition. It is emphasized that this design procedure has been developed for use in aggresive operational conditions - application to a less difficult situation could result in an overdesign.

The procedure is illustrated in fig b1. (page 126).

B.1 GRAIN-SIZE RANGE

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

Range A : 40 % or more of the soil particles will be smaller than or equal to 0.06 mm

Range B : 15 % of less of the soil particles will be smaller than or equal to 0.06 mm.

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

This is represented diagramatically in fig. b2. The vertical line for grain-size equal to 0.06 mm is divided into three portions A, B and C. Portion A lies above 40 %, portion B lies below 15 % and portion C lies between 15 % and 40 %. The grading curve for soils falling into range A will either lie wholly to the left of this vertical line or pass through portion A of the line.

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

Curves falling into range C will pass through portion C of the line.

170

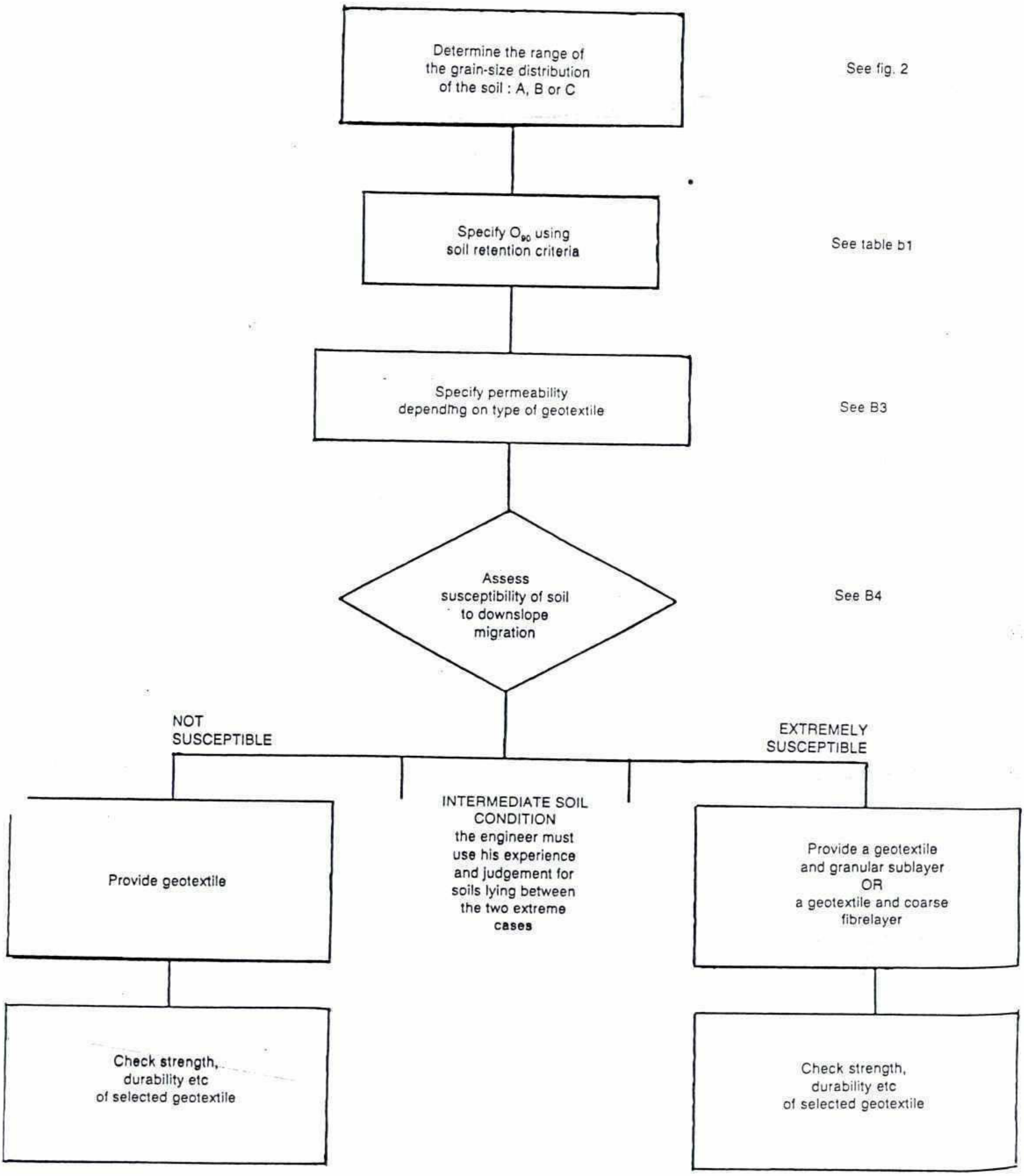


FIG. b1 : Specific design procedure for a geotextile filter

In fig. b2 some typical grading curves are given as examples of these ranges.

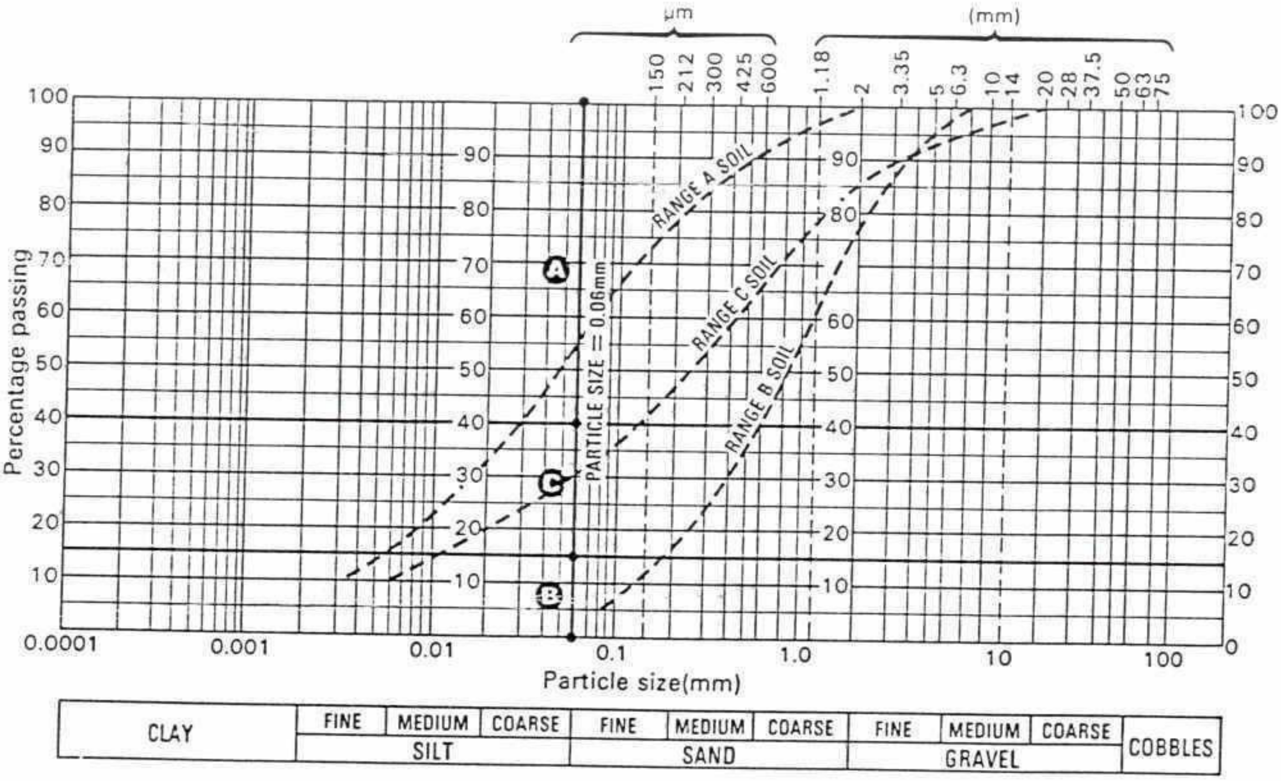


FIG. b2 : Soil grading ranges

B.2 DESIGN FOR SOIL RETENTION

Once the range has been established the filter can be designed for soil retention. The criteria for soil retention are based on the value of the effective opening size 0_{90}. A knowledge of the test method used to determine O_{90} is vital when designing the filter.

The following rules given in table b1 can only be used for 0_{90} values determined by the wet seiving analysis of the Franzius Institute for Hydraulic Research and Coastal Engineering. This testing method is laid down in Swiss standard SN640550.

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

(See Fig. b1 on page 128.)

B.3 DESIGN FOR PERMEABILITY

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

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

172

GRAIN SIZE RANGE	RETENTION CRITERIA
A	$0_{90} < d_{90}$ * $< 10\ d_{50}$ < 0.3 mm
B	$0_{90} < 1.5\ d_{10}\sqrt{Cu}$ $< d_{50}$ < 0.5 mm
C	As range B

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

$$0_{90} < 2\ d_{90}$$

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

TABLE b1 : Soil retention criteria

In general the permeability criterion requires that,

$$\eta\ k_g \geqslant k_s$$

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

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

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

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

2. For woven fabrics the reduction factor is dependent upon the permeability of the geotextile on the d_{10} of the soil, and can be found from fig. b3.

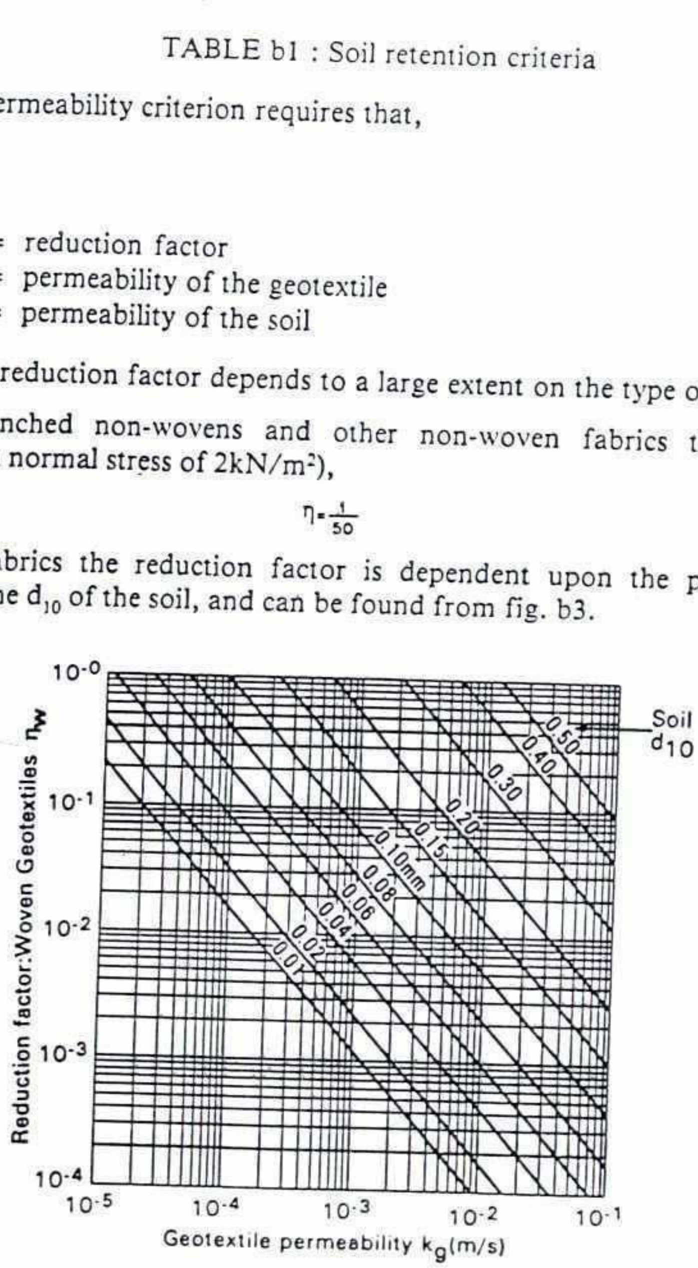


FIG. b3 : Permeability reduction factor

3. For thin non-woven geotextiles no specific criteria are available. However the filtering behaviour of these fabrics is, to some extent similar to woven fabrics and it is therefore suggested that fig. b3 is used if no other information is available.

B.4 IDENTIFICATION OF SOILS SUSCEPTIBLE TO DOWNSLOPE MIGRATION

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

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

The following procedure gives a general method of identifying soils susceptible to downslope migration. There may however be exceptions to the conditions given here and the designer must use his own judgement in such cases; see also table 3.4 (a) and (b).

Soils susceptible to downslope migration will satisfy the following condition,

A proportion of particles must be smaller than 0.06mm.

Additionally the soil will satisfy at least one of the following,

1. The coefficient of uniformity, $$C_u = \frac{d_{60}}{d_{10}} < 15$$

   The limit of 15 is a conservative estimate based on practical experience. For soils with a coefficient of uniformity greater than 15 a secondary filter will normally become established.

2. 50 % or more of the particles will lie in the range 0.02 mm $< d <$ 0.1 mm.

3. The plasticity index

   $$I_p < 0.15$$

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

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

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

B.5 METHODS OF PREVENTING DOWNSLOPE MIGRATION

There are three ways of preventing serious downslope migration.

(a) ATTENUATION OF THE HYDRAULIC GRADIENT

This can be achieved by incorporating a granular sublayer between the geotextile and the coverlayer. This is discussed in 3.5.3.1. In practice the granular material should be fine enough to provide an adequate damping effect yet coarse enough to be retained by the cover layer. Further details are given in 3.6.5.

174

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

(b) STABILIZATION OF THE SOIL SURFACE

Experience has shown that a thick layer of coarse fibres attached to the back of the geotextile filter integrates with the soil surface, thus reducing downslope migration of soil particles within this layer (plate b1).

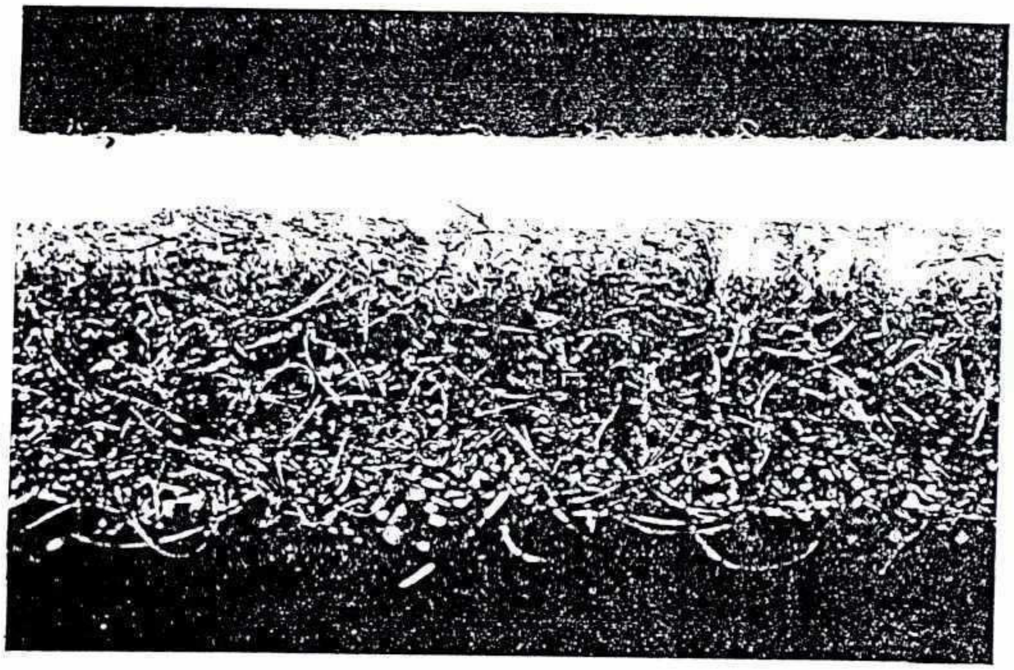


PLATE b1 : Geotextile incorporating additional coarse layer

The physical requirements of this layer are dependent upon the range of the soil grading curve (3.6.4.1.) and are given in table b2.

CHARACTERISTIC OF COARSE LAYER	RANGE A GRADING CURVE	RANGE B GRADING CURVE
O_{90}	$0.3 < O_{90} < 1.5$ mm	$0.5 < O_{90} < 2.0$ mm
Thickness	$5 < t_{fg} < 15$ mm	$5 < t_{fg} < 20$ mm

TABLE b2 : Requirements of the coarse layer

As well as these requirements experience has shown that the filter fabric should have a minimum thickness of about 5 mm.

(c) APPLICATION OF A CONSTRAINING LOAD

This can by achieved by using a heavy weight coverlayer which imparts a significant load into the bank slope thereby resisting uplift pressures caused by excess pore water pressures.

175

This effect is demonstrated on the Mohr-Coulomb diagram given in 3.5.2.4; Appendix A suggests a calculation method which could be adopted to find the weight of coverlayer. However at present, research is still being carried out in this area and as a practical means of preventing downslope migration either alternative (a) or (b) is suggested.

Fig. b4 illustrates alternatives (a) and (b) together with the case where the soil is not susceptible to downslope migration.

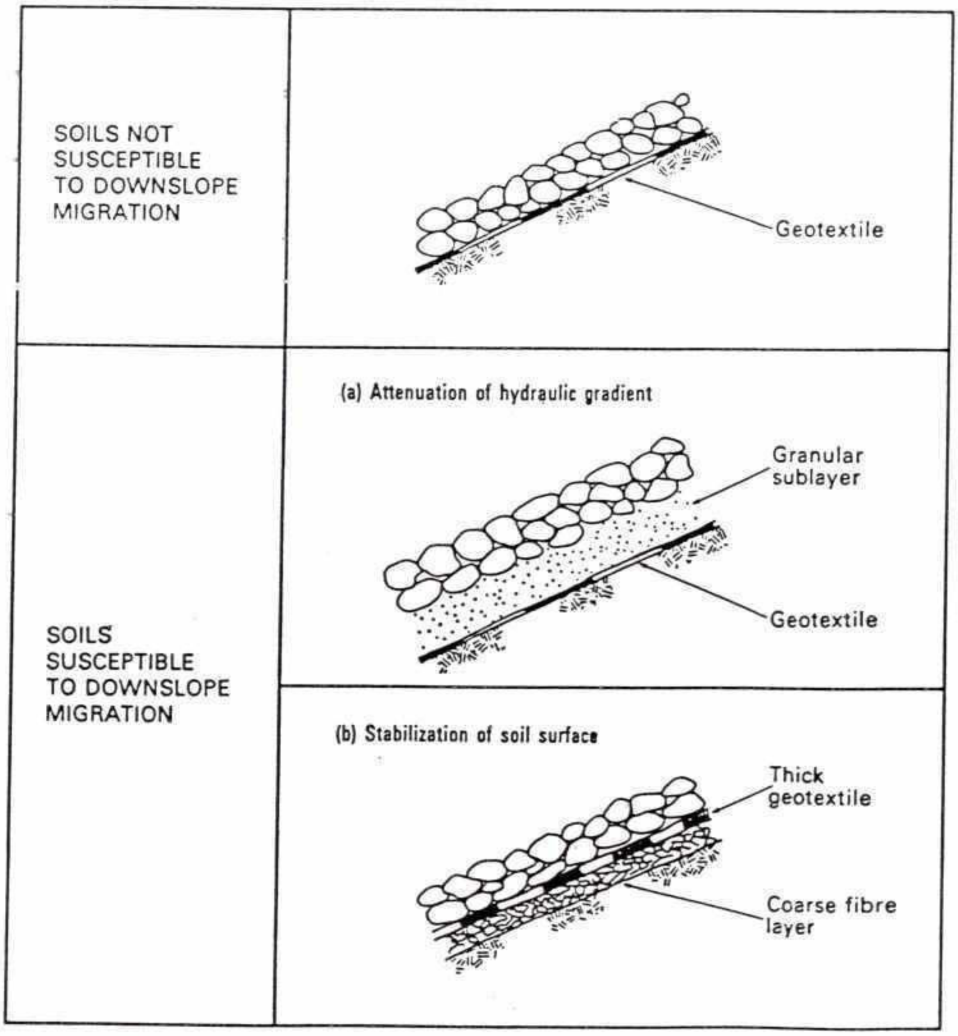


FIG. b4 : Recommended filter systems

B.6 DESIGN ASPECTS OF SOILS NOT SUSCEPTIBLE TO DOWNSLOPE MIGRATION

For soils not subject to the problem of downslope migration it is necessary for the filter to only meet the requirements of B.2 and B.3.

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

B.7 WORKED EXAMPLE

Consider the following soil,

$d_{10} = 0.002$ mm $k_s = 10^{-4}$ m/s
$d_{50} = 0.021$mm
$d_{60} = 0.032$mm
$d_{90} = 0.12$mm

1. Grain size range.
   Plotting these points on fig. b2 the grading curve falls in range A.

2. Effective opening size of geotextile.
   Referring to table b1.
   $d_{90} = 0.12$ mm
   $10d_{50} = 0.21$ mm
   $O_{90} < 120$ μm

3. Permeability of fabric (non-woven)
   $k_g > 50\ k_s$
   $k_g > 5 \times 10^{-3}$ m/s

4. Identification of susceptibility to downslope migration.
   Are a proportion of the soil particles smaller than 0.06 mm ?
   YES
   Using condition (3) in B.4.
   $\frac{\text{proportion of clay}}{\text{proportion of silt}} = \frac{10}{65} = 0.15 < 0.5$
   soil is susceptible to downslope migration

5. To prevent downslope migration use an additional coarse layer beneath the fabric.
   Using table b2 the coarse layer must have the following characteristics,
   $0.3\text{mm} < O_{90} < 1.5\text{mm}$
   5mm < thickness < 15mm

APPENDIX F

ASSESSMENT OF MAXIMUM NEAR BANK VELOCITY

ASSESSMENT OF MAXIMUM NEAR BANK VELOCITY

1. Outline

As mentioned above, the river is highly braided within the study area. Therefore the flow patterns are highly complex and velocities variable. A number of studies were undertaken to derive the maximum velocities that are likely to occur in the river at the planned bank protection and river training structures. The design of structures has been based on the maximum near bank velocity with a 100 year return period. The value of this parameter is derived in the following manner (Figure F1).

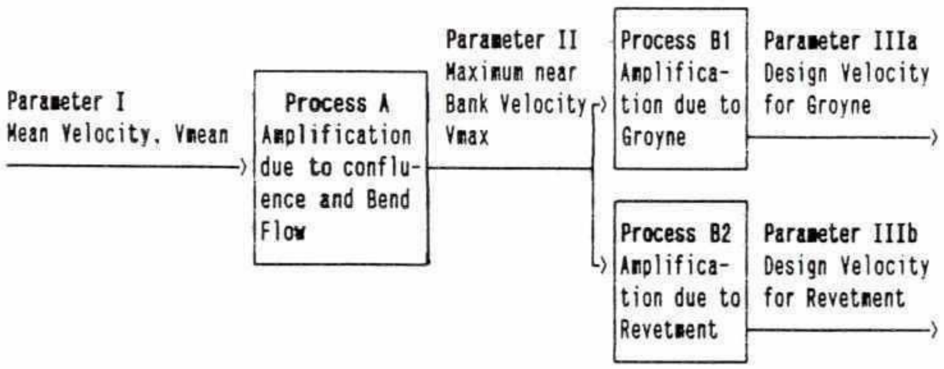


Figure F 1: Parameters and Processes for Assessment of Design Velocities

The procedure steps are:

(a) Parameter I, V_{mean}, is found from an assessment of the distribution of mean velocities in the river for 100 year flow conditions.

(b) Process A is the amplification of flow velocity due to confluence or bend flow or a combination thereoff.

(c) Parameter II is the resultant maximum velocity for undisturbed flow (i.e. no structures present).

(d) Processes B1 & B2 are the amplification of flow velocity near structures. B1 refers to the presence of groynes while B2 refers to revetment type structures.

(e) Parameters IIIa & IIIb are the resultant near structure velocities in the case of groynes and revetments.

The derivation of the parameter values has involved the use of numerical and physical modelling in addition to the interpretation of morphological data, as outlined below.

2. Parameter I, Distribution of Mean Velocity in the River

The distribution of mean velocity in the river has been derived from analysis and interpretation of the following three data sets.

o MIKE11, 1-D modelling giving the mean at-a-section velocity along the Jamuna for varying flow.

o All available river cross-sections from which the variation of cross-sections with time, and thus the velocity, can be computed.

o BWDB discharge data at Bahadurabad for the period 1969 - 1990.

2.1. 1-D Modelling

Figure F 2 shows the variation with chainage of the mean velocity, V_{mean}, found using the BRTS 1-D hydrodynamic model of the Jamuna. The velocities are for the peak of the 1988 flood which was close to the 100 year condition. Considerable scatter is apparent but no obvious trend in the velocity in a longitudinal direction. The mean value is found to be 1.46 m/s and the standard deviation 0.43 m/s. This variability may be explained in terms of the passage of massive sand bars that result in substantial variation in cross-sectional area along the length of the river. In Figure F 3 the probability and distribution functions are shown, assuming a normal distribution, with these parameters.

2.2. Analysis of Cross-sectional Data

An analysis of all BWDB cross-sectional data for the period 1964/65 to 1988/89 has been carried out. To obtain flow velocities, the discharge along the river for the 100 year return period was used based on water levels derived from the 1-D modelling. The average velocity was found to be 1.58 m/s and the standard deviation 0.5 m/s. It appears by comparisons with the data derived from the MIKE11 simulations that the average velocity and the standard deviation are both slightly higher. This reflects the additional variability of cross-sectional area over time.

2.3. BWDB-Discharge Data for Bahadurabad

Measured mean velocity and water level (WL) data for Bahadurabad are available from BWDB. The data is for the period 1969 to 1990. By extrapolation to the 100-year WL it was found that the sectional mean velocity falls in the range of 1.4 m/s to about 2.0 m/s and that the mean value is about 1.65 m/s.

Given that Bahadurabad is only one station, there appears to be reasonable agreement between these results and the other results from 1-D modelling and cross-sectional data. In view of the consistency between these three sources, the data derived from the 1-D modelling has been adopted on the grounds that it is the most systematic and amenable to further statistical analysis.

3. Process A, Amplification due to Bend and Confluence Flow

It has been seen that the mean velocity of the river varies substantially from cross-section to cross-section. In addition, when considering any specific cross-section, the velocity varies over the cross-section. These variations are due to the highly complex variation in flow flux due to the shifting flow channels and chars. When bends or confluence of two branches occur, this background variation is accentuated.

In order to derive maximum near bank velocities required for the design of river training works, physical model studies using the Fulcharighat, Sariakandi, Kazipur and Sirajganj model beds were used. By measuring scaled velocities for the base cases and a range of intervention options it was possible to derive a set of factors relating maximum near-bank velocity to mean sectional velocity. Amplification factors in the range 1.50 to 2.15 were found.

The base cases provided data on the velocity variation for naturally occurring conditions as represented by these particular bathymetries, ranging from "normal" to "severe". There is insufficient data for a rigorous statistical approach for the derivation of frequency of occurrence of these different degrees of severity but from insight gained through the morphological and other studies it is possible to make the following reasoned assessments:

(a) The 1-D model contains about 100 cross-sections and the situation at Kazipur represents the largest amplification found in the study area. Thus the maximum recorded amplification of 2.15 at Kazipur is considered to have approximately a one percent probability of exceedance in a given cross-section.

(b) The smallest amplification factor of 1.5, found at Sariakandi is very typical for "normal" situations when no significant bend or confluence is present. This value is taken as the mean amplification factor.

Thus, assuming the amplification factor follows a normal distribution, the mean amplification may be taken as 1.50 and the standard deviation is 0.28.

4. Parameter II - Resultant Near-bank Velocity Distribution (100 year return period)

By combining the results for parameter I and the process A (amplification) the resultant distribution of near bank velocities for a 100 year return period has been estimated. The results presented in Figure F 4 indicate that the mean maximum velocity is 2.25 m/s and the 10 percent and 5 percent exceedance velocities are at 3.2 and 3.6 m/s respectively. It is important to note that this maximum velocity is associated with features that may be located near the right or the left bank or even away from the bank. Therefore, in assessing the probability of occurrence in relation to one bank only, it is reasonable to reduce the total probability by a factor 0.5, or possibly 2greater.

5. Process B1 & B2 - Amplification due to Structures

The physical model studies provided the means for measuring the velocity amplification due to the introduction of training works.

Velocity at Groyne Noses

It was found that a groyne will typically amplify the velocity by a factor of 1.4 +/- 10 percent. This may be stated as an amplification factor of 1.40, with standard deviation of 0.14 and assuming again that a normal distribution applies.

Velocity at Revetment Terminations

The extent of any amplification is dependent on the upstream approach conditions. In the case of a revetment which is not protruding at all, i.e. stabilising an otherwise straight river bank, the amplification factor will be close to 1. In the case of a protruding structure the amplification factor was found to increase up to about 1.3 +/- 10 percent.

For the design of revetments a factor of 1.1 has been adopted for straight sections to take into account the fact that some amplification can occur if a flow channel develops with deep water meeting the revetment at an angle. For the up-stream termination of a hard-point an amplification factor of 1.3 is being used.

6. Parameters IIIa and IIIb. Resultant near Bank Velocity Distribution in the Case of Structures (100 year return period).

Using this approach the resulting parameters IIIa and IIIb are computed as shown in Table 1.

Table 1: Maximum Velocities (m/s) for given Exceedance Probability for a 100 year Return Period.

Parameter	Exceedance Probability (%)					
	2	5	8	10	20	50
U-mean (m/s)	2.3	2.2	2.1	2.0	1.8	1.5
U-max (m/s)						
no structure	4.0	3.6	3.3	3.2	2.8	2.2
groyne (1.4)	5.7	5.1	4.8	4.6	4.0	3.0
revetment (1.1)	4.5	4.0	3.7	3.6	3.1	2.3
revetment (1.2)	4.8	4.3	4.1	3.9	3.4	2.6
revetment (1.3)	5.2	4.7	4.4	4.2	3.7	2.8

Note: The figures in brackets denote the mean amplification factor applied.

V67

7. Application to Training Works Design

Given that the design criterion is that there should be no more than a 1 percent risk of the design velocity being exceeded during a 30 year nominal structure life, the values is in Table 1 should be interpreted as follows.

For a project life of 30 years there is a probability of 26 percent for the occurrence of a 100 year hydrological event. Thus for a risk of 1 percent over the project life, the velocities in Table 2 for 8 percent exceedance probability should be used (i.e. 0.08 x 0.5 x 0.26 = 1 percent). The factor of 0.5 being applicable because the maximum velocity can only occur near one or the other bank (or an island)

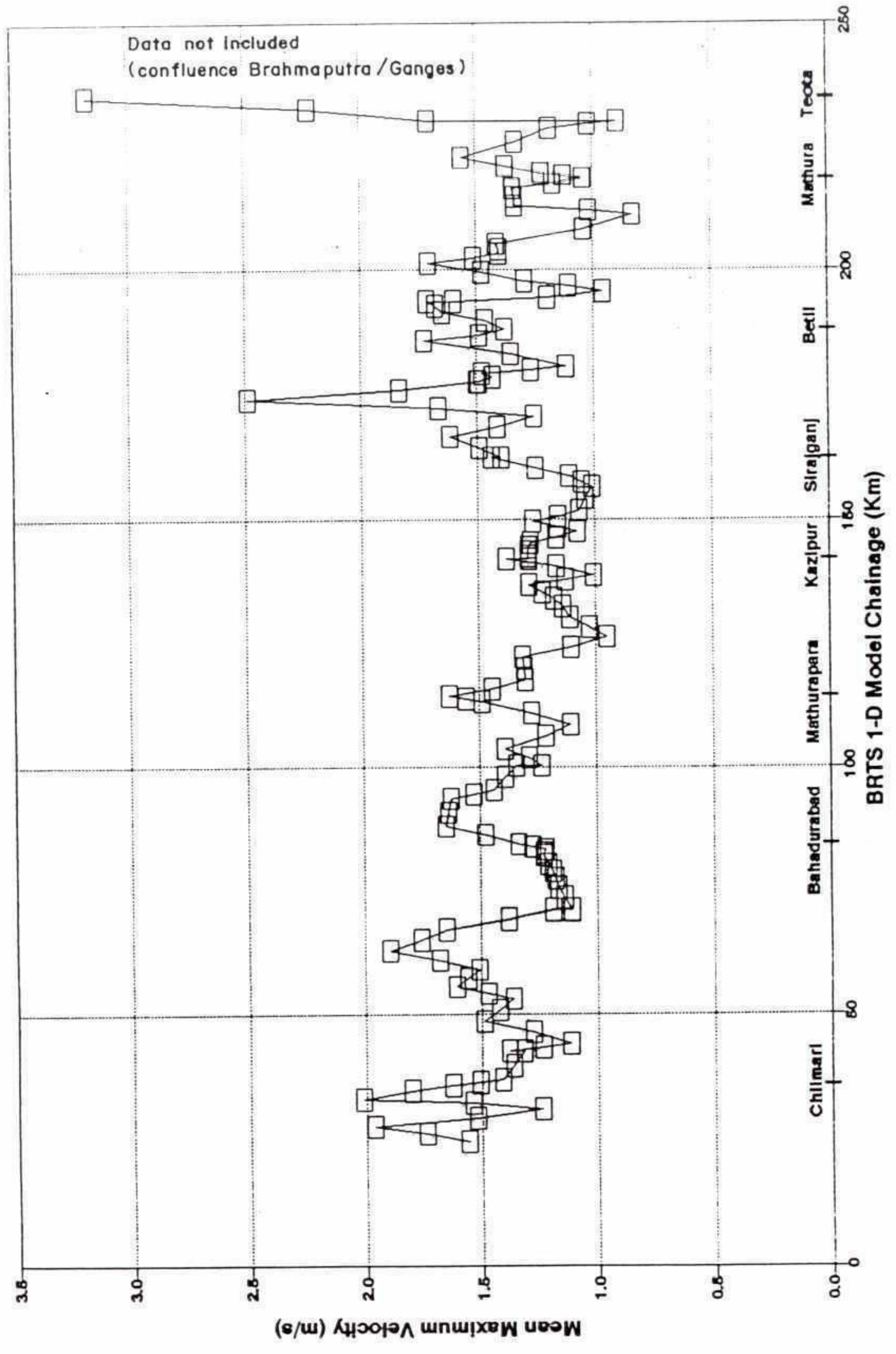
Table 2 shows the derived design velocities that are being used for phase 1 of the Brahmaputra stabilisation works.

Table 2: Design Velocities for different Structural Components

Type of Structure	Amplification Factor	Design Velocity (m/s)
Revetment Straight Section	1.1	3.7
Revetment Up-stream Termination	1.3	4.4
Head of Groyne	1.4	4.8

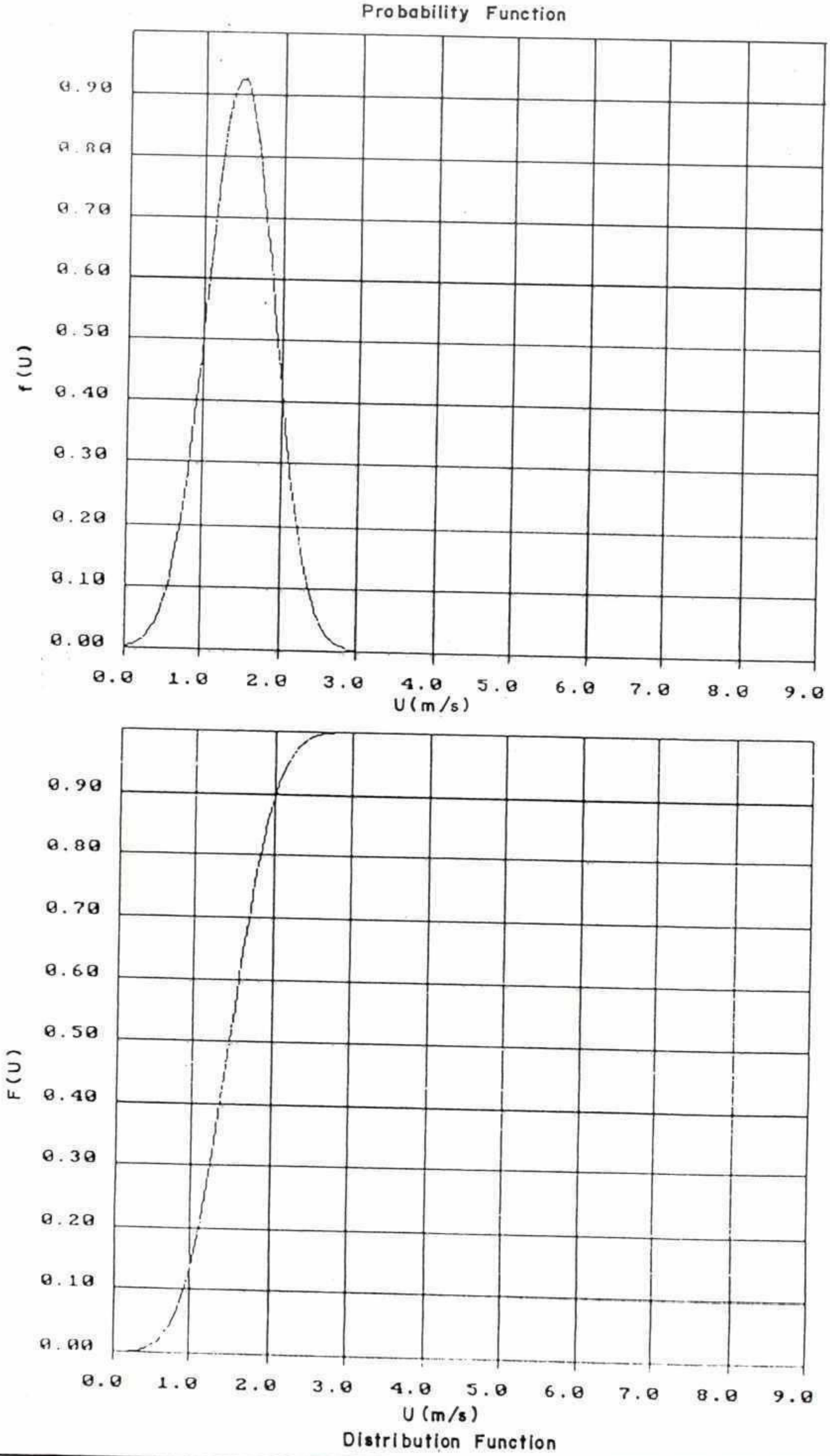
Note: Based on a 1 percent probability of exceedance of design conditions in the project life (30 years).

182



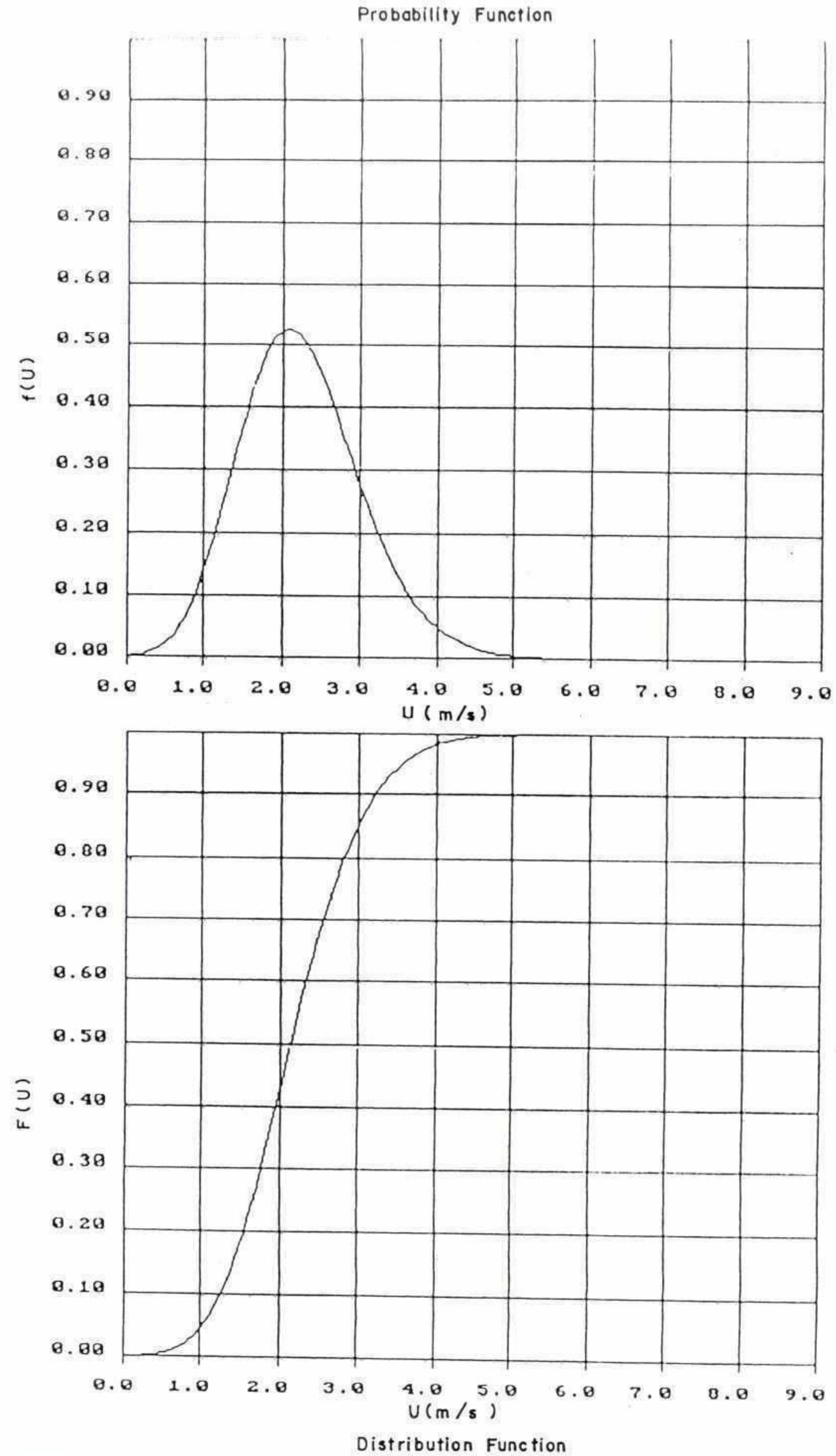
Mean Maximum Velocities (1988) vs. Model Chainage

	Annex 4 · Appendix F
Halcrow/DHI/EPC/DIG *May 1993*	Figure F.2



Probability and Distribution Functions for Mean Velocity from Mike 11 Simulations

Halcrow/DHI/EPC/DIG	May 1993	Annex 4 Appendix F Figure F.3



Probability and Distribution Functions for Maximum Velocity from Mike 11 Simulations

	Annex 4 Appendix F
Halcrow/DHI/EPC/DIG *May 1993*	Figure F 4

185

APPENDIX G

QUARRY ROCK ARMOURING (ALTERNATIVE)

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

Quarry Rock Armouring (Alternative)

The size of quarry rock is determined from the JMBA equation:

$$D = \frac{0.7\, v^2}{2\,(S_s-1)g} \cdot \frac{2}{\left[\log \left(\frac{6h}{D}\right)\right]^2} \cdot \frac{1}{\left[1-\left(\frac{\sin \phi}{\sin \theta}\right)^2\right]^{1/2}}$$

where =

D = D_{50} of rock material [m]

v = Maximum near structure velocity = 3.7 m/s

S_s = Specific gravity of rock material (assumed at 2.60)

g = acceleration of gravity = 9.81 m/s²

h/D = water depth/rock size, taken at 5.0 as for concrete block design.

ϕ = bank slope in degrees
(for 1:3.5 slope, $\phi = 15.95^0$, $\sin \phi = 0.274$)

θ = angle of friction of rocks = 40^0, $\sin \theta = 0.642$

From this

$$D = \frac{0.7\, v^2}{2x1.6x9.81} \times \frac{2}{2.18} \times 1.106$$

$$D = 0,023\, v^2$$

This yields

$$D_{50} = 0.023 \times 3.7^2 \approx 0.32 \text{ m}$$

The parameter D_{50} is defined as:

$$D_{50} = \left(\frac{\bar{W}}{s} \right)^{1/3}$$

where =

$\bar{W}$ = average weight of rock armour (t)

s = density of rock (t/m³)

Thus the "average" rock weight is found:

$\bar{w} \quad = \quad {}_s \times D_{50}^{3} = 0.085$ t (85 kg)

The "average" volume, V, is given by:

$V = D_{50}^{3} = 0.033\ m^3$

The layer thickness should as a minimum be taken as two layers randomly placed. The nominal layer thickness is calculated from:

$r \quad = \quad n.c.\ V^{1/3} = n.c.D_{50}$

$n \quad = \quad$ number of layers

$c \quad = \quad$ shape factor = 1.0 for quarry rock

The layer thickness is then found:

$r \quad = \quad 2.1 \times 0.32 = 0.64$ m

The number of rocks per m^2 is determined from:

$$N \quad = \quad n.c\ \left(1 - \frac{P}{100}\right)\ V^{-2/3}$$

$P \quad = \quad$ porosity of rock layer = 40 %

$N \quad = \quad 2.1 \times 0.6 \times 0.033^{-2/3} = 11.66$ rocks/m^2

And the weight of quarry rock is equal to:

Weight = 11.66×0.085 t = $0.99 \approx 1.0$ t/m^2*

Grading of Rock:

Concrete blocks are manufactured and have consequently almost uniform size and weight. For quarry rock the situation is different and some spread/variation in size will be allowed.

Practical experience shows that for sorting in a quarry a range of about ± 20 % on D is feasible. This means the following range of sizes:

$0.27 \leq D \leq 0.39$ m

The weight range is then found:

50 kg $\leq W \leq$ 150 kg

* Note Metric [t]

The same methodology leads to the rock sizes required for upstream terminations and noses of groynes thus:

Upstream Terminations:

v = 4.4 m/s

D_{50} = 0.45 m

Average weight = 240 kg

Nominal weight range = 140 kg to 425 kg

Noses of Groynes

v = 4.8 m/s

D_{50} = 0.53 m

Average weight = 385 kg

Nominal weight range = 225 to 680 kg.

APPENDIX H

WAVE CHARACTERISTICS

4. WAVE CHARACTERISTICS

4.1 Introduction

This chapter is concerned with the derivation of design wave characteristics for the design of bank stabilisation revetment on the right (west) bank of the Brahmaputra River.

4.2 Description of the Problem

The river training and bank protection works at the priority sites will have to be designed taking into account the environmental loads that are likely to occur within the lifetime of the structures in question. The environmental loads comprise currents, water levels & waves. Currents and water levels are covered by other BRTS studies, while the study of waves is limited to the present assessment. The river training works and bank protection revetments are designed to withstand the maximum currents occurring during the lifetime of these structures and the size of concrete blocks are selected to be able to resist the current action.

However strong winds may occur causing wave action. It is therefore of importance to analyse the wind conditions and the likely size of such waves and their occurrence. The fetch for generation of waves varies significantly over the year with the water level, so it is not only of importance to evaluate the wind conditions but also at what time of the year they occur.

4.3 Winds in the Interior of Bangladesh

Strong winds in the interior of Bangladesh along the Right Bank of the Brahmaputra River occurs due to the following two phenomena:

(a) Squalls

Squalls are small local disturbances associated with the occurrence of thunderstorms. They are also locally named "North-Westerns". Squalls mainly occur in the pre-monsoon season from March to May, but can also occur at other times of the year.

Typically a squall has a rapid rise in wind velocity followed by a turn in the wind direction after which the wind velocity gradually decays. The whole duration of a squall is normally limited to $^1/_2$ to 1 hour. The Jamuna Bridge Reports present the following distribution curve for the wind velocity, V, (peak gust speed) on a thunderstorm as function of the Return Period, T. V is in [m/s] and T in [years].

$$V_{thunderstorms} = 30 + 4.5 \times \ln (T) \text{ [m/s]}.$$

(b) Cyclones (Typhoons or Hurricanes)

Cyclones such as the one that occurred late April 1991 are associated with very strong wind velocities. These cyclones occasionally strikes the coastal areas of the Bay of Bengal. However the development and travelling of a cyclone is dependent upon a constant supply of

191

moisture for which reason a cyclone normally weakens when entering in over land areas. This is known from studies in many countries all over the world and especially from the occurrence of cyclones in the Caribbean and on the East Coast of USA. Cyclones are called hurricanes in the USA.

In Bangladesh cyclones moving in over land are thus having decaying wind speeds relative to the conditions over the sea. Cyclones in the central to northern part of Bangladesh along the Brahmaputra River are rare according to the Bangladesh Atlas. According to the studies performed for the Jamuna Bridge a severe cyclone will only come within a range (i.e. 40 km from the centre) about once in 30 years.

The Jamuna Bridge Report further states that the associated gust speed would be 33 m/s. The Return Period for maximum gust speeds is related to the gust velocity by:

$V_{cyclone} = 8 + 8 \times \ln(T)$ [m/s].

From these distributions, it is found that for Return Periods less than 539 years the squalls gives the highest wind speeds.

The above distributions were worked out for the Jamuna Bridge which is close to the Southern limit of the BRTS project.

It is important to mention that the distributions were for gust wind speeds and not for sustained winds. Normally the wind data relates to 10 min average values measured 10 m above the ground - level.

Typically for normal long duration storms there is a factor of about 1.5 between the gust and the average wind speeds. The Jamuna study reports that a gust speed of 33 m/s corresponds to approximately 20 m/s. For a squall the relation between the peak gust wind speed and the average 10 min wind is more variable according to the Jamuna Bridge Report.

The BRTS project covers the right embankment of the Brahmaputra River up to the Teesta River. From the previous description, it is clear that the cyclone intensity and associated wind speeds are decreasing when moving towards north.

Therefore in conclusion squalls (North Westerns) are considered to be the determinant factor for extreme waves in the Brahmaputra River for all sites from say Sirajganj and further to the north.

4.4. Analysis of wind data

4.4.1 Data

Wind data has been supplied by the Bangladesh Meteorological Department in Dhaka.

The data has been analysed with the aim of obtaining further insight into the extreme wind conditions prevailing along the Brahmaputra River. Data from the following meteorological stations has been collected.

a. Faridpur
b. Sirajganj
c. Bogra

d. Rangpur
e. Mymensingh

The data from the stations are of variable quality and coverage. Many obvious errors were identified in the data material at the first review. Upon identification of these errors - the data was corrected by the Meteorological Department. It cannot however be excluded that further errors are present. The data from Bogra and Rangpur covers the most extensive period with fewest periods without data. The data covers 10 min average winds (speed in knots and direction in degrees).

4.4.2 Conclusion of Wind Data Analyses

The analyses performed on the wind data from the five meteorological stations show reasonable consistency in the occurrence of high average wind speeds exceeding 22 knots (Tables 4.4 - 4.8). The following main conclusions have been reached.

a. Wind situations with wind speed exceeding 22 knots do not occur every year. The frequency of occurrence appears to be in the range once per 2-4 years.

b. The events with high wind speeds are most frequent in the pre-and post monsoon periods, but high winds are also reported in January, February, and in the monsoon months of June, July, August, September.

c. The maximum wind speeds equal to or exceeding 30 knots recorded are as follows:

-	35 knots, direction	340^0	(Faridpur)
-	30 knots, direction	150^0	(Faridpur)
-	35 knots, direction	130^0	(Sirajganj)
-	30 knots, direction	200^0	(Bogra)
-	30 knots, direction	360^0	(Bogra)
-	30 knots, direction	130^0	(Bogra)
-	30 knots, direction	50^0	(Mymensingh)

It appears that the maximum winds can occur from almost all directions.

d. The data given covers about 18 years for most of the stations, and a wind Speed of 35 knots occurred two times. This event can thus be assessed to have a return period of about 5 years considering that there were many interruptions in the data collection. 35 knots are equal to 18.0 m/s.

e. The Jamuna Bridge Report yields for a 10 years Return Period a peak gust speed of, $V = 30 + 4.5 \ln 5 = 37.2$ m/s. Using the conversion factor of $20/33 = 0.61$, this corresponds to 22.5 m/s which is higher than the about 18 m/s found in the present analyses. (25 % higher).

f. On this basis the Jamuna Bridge wind statistics is considered to yield conservative results.

g. For design of the works a 50 years return period of wind velocity will be used in the following applying the Jamuna Bridge statistical distribution reduced by 25 %. The 50 years average wind speed is than $V = 0.8 \times 0.61 \times (30 + 4.5 \ln 50) = 23$ m/s.

h. It is important to notice that all the structures to be built on the right bank of the river will be approximately in the direction North - South. This means that they will only be directly exposed for winds from directions in the angle 45^0 to 135^0 approximately.

Three out of the seven events reported above is within this sector, i.e. less than 50 %. Therefore the assessment of Return Periods is somewhat conservative. A factor of two on the Return Period in the Jamuna Bridge Formula results in a change in the wind speed of 7 %, i.e. that 23 m/s is reduced to 21.3 m/s. In conclusion and considering the uncertainties the design wind speed is fixed in the range 20 to 25 m/s, with an estimated central estimate of 22.5 m/s.

i. The Jamuna Bridge Reports presents an example of Anemometer Recordings from Squalls.

It is shown in Fig. 4.1. It appears that the squalls are of very short duration. The peak is lasting only from a few minutes up to about 20 min. Therefore in the following a duration of 30 min is considered to be on the safe side for assessing wave heights.

4.5 Water Level and Fetches for Wave Generation

4.5.1 Water Levels

Another important parameter for the generation of waves is the fetch. The fetch is the distance over the water on which the wind blows and the waves are generated. The Brahmaputra River is braided and only at the peak of extreme monsoon floods the chars are flooded and a large fetch is present for wave generation. It is therefore important to consider the water level variations in the river and the corresponding cross-sections and derived fetches.

The available data on mean water levels in the BRTS Interim Report No. 1 has been analysed to obtain average water levels as function of site and month of the year. Three sites have been analysed. The results appear in Table 4.1.

Table 4.1 Average Water Levels for Three Sites for all Months of the Year (m) PWD)

SITE	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR
Fulchari	14.3	15.7	17.5	18.8	18.7	18.3	17.1	15.0	14.0	13.5	13.3	13.4
Kazipur	9.1	10.8	12.6	14.0	14.0	13.5	12.3	10.1	9.0	8.3	7.8	8.1
Sirajganj	8.0	9.7	11.6	13.0	13.1	12.8	11.4	9.2	8.0	7.2	6.9	7.1

4.5.2 Typical Cross-Sections

Fig. 4.2 shows the position of three selected cross-sections for the three sites. The soundings are from 1985-86. The cross-sections are shown in Figs. 4.3 - 4.5.

The order to obtain an estimate of the average fetches three water levels have been considered on each of the cross-sections.

These are:

High Stage: This water level corresponds to a typical water level in the months JUL, AUG, & SEP.

Medium Stage: This water level corresponds to a typical water level in the pre-and post monsoon period.

Low Stage: This water level corresponds to a typical water level in the dry season DEC-MARCH.

4.5.3 Assessment of Fetches

The following assessment of fetches (Direction East - West) has then been made (See Table 4.2)

Table 4.2 Maximum Fetches [m]

Site	Water Level Stage		
	High	Medium	Low
Fulchari	3.350	1.700	1.500
Kazipur	2.750	2.600	1.300
Sirajganj	1.700	850	500
Maximum Fetch (1.5xMax.Fetch)	5.000	3.900	2.300

It appears that the fetch in the east-west direction are limited. To take into consideration that the wind can blow from other directions ($\pm$ 45^0) relative to East these maximum fetches are increased by a factor of 1.5. The results appear in Table 5.2 showing fetches ranging from 2.300 m in the dry season to about 5.000 m in the high Flow Stage.

4.6. Wave Calculations

The data on winds and fetches in the previous sections can now be used to derive wave conditions (wave height, Hs and Peak Wave Period, Tp).

For this purpose the paper "Revision of SPM 1984 Wave Hindcast Model to Avoid Inconsistencies in Engineering Applications" has been used.

The results of the assessment appear in Table 4.3. All results are for a wind duration of ½ hour.

125

Table 4.3 Calculation of Wave Heights and Periods
U = 22.5 m/s.

Season (WL-Stage)	Fetch [m]	Depth [m]	Significant Wave Height Hs [m]	Peak Wave Period Ts[s]
High	5.000	5.0	1.00	3.0
Medium	3.900	7.5		
Low	2.300	10		

In conclusion considering that most of the strong winds occur in the pre- and post monsoon periods a "design" wave height of Hs = 1.0 m and a peak wave period of, Tp = 3.0 s is selected for design purposes.

It should be noted that this assessment is not taking the current into consideration. The current velocity in the river is large and the flow is highly turbulent. This is believed to have a damping effect on the short waves. It is not possible with available theories to quantify this effect.

Table 4.4 Maximum Wind Speeds Faridpur

Station: FARIDPUR

Data Coverage: JAN 1970 to DEC 1989

Data Details :	
JAN 1970 to FEB 1971	DATA ONLY 00, 03 & 12 H
MAR 1971 to JUN 1971	NO DATA
JUL 1971 to DEC 1972	DATA ONLY 00, 03 & 12 H
JAN 1973	NO DATA
FEB 1973 to MAY 1975	DATA ONLY 00, 03 & 12 H
JUN 1975 to NOV 1976	DATA 00, 03, 06, 09 & 12 H
JUL 1978	NO DATA

Maximum Events With Speed > 22 knots

Maximum Wind Speed and Directions are presented below

Year	Month	Date	Hour	Speed Knots	Dir. deg.
1971	SEP	30	00	23	110
1977	MAY	13	00	25	90
1986	MAR	31	15	35	340
1986	SEP	26	15	23	90
1988	APR	14	15	24	140
1988	APR	15	09	24	180
1988	MAY	25	09	24	190
1988	MAY	25	12	24	190
1988	MAY	25	18	26	130
1988	AUG	02	21	24	90
1988	OCT	19	09	23	10
1988	NOV	30	00	25	330
1989	APR	23	12	23	180
1989	MAY	27	03	24	150
1989	MAY	27	06	30	150
1989	MAY	27	21	23	150
1989	JUL	29	03	26	250
1989	JUL	29	06	25	250
1989	JUL	29	09	25	280

137

Table 4.5 Maximum Wind Speeds Sirajganj

Station: SIRAJGANJ

Data Coverage: JAN 1961 to DEC 1980

Data Details :	JAN 1961 to FEB 1965	DATA 00, 03 & 12 H
	MAR 1961 to JUN 1966	DATA 00, 03, 06, 09 & 12 H
	JUN 1966 to MAR 1968	DATA 00, 03 & 06
	MAR 1968 to JUN 1969	DATA 00, 03, 06, 09 & 12 H
	JUN 1969 to MAR 1970	DATA 00, 03 & 12 H
	MAR 1970 to JUN 1970	DATA 00, 03, 06, 09 & 12 H
	JUN 1970 to FEB 1971	DATA 00, 03, 06 & 12 H
	FEB 1971 to AUG 1971	NO DATA
	FEB 1971 to DEC 1971	DATA 00, 03 & 12 H
	DEC 1971 to MAR 1971	NO DATA - OR SCARSE DATA
	APR 1971 to FEB 1975	DATA 00, 03 & 12 H
	FEB 1975 to JUN 1975	NO DATA
	JUN 1975 to SEP 1975	DATA 00, 03 & 12 H
	OCT 1975 to JUN 1978	NO DATA
	JUN 1978 to DEC 1980	DATA 00, 03 & 12 H

Maximum Events With Speed > 22 knots			Maximum Wind Speed and Directions are presented below		
Year	Month	Date	Hour	Speed Knots	Dir. deg.
1963	JUN	7	12	35	130
1969	APR	4	12	23	180

Table 4.6 Maximum Wind Speeds Bogra

Station: BOGRA

Data Coverage: JAN 1970 to DEC 1988 both months included

Data Details :	26 MAR 1971 to 30 JUN 1971	NO DATA
	01 JAN 1977 to 30 JUN 1971	NO DATA
	01 FEB 1978 to 28 FEB 1978	NO DATA
	28 OCT 1978 to 31 DEC 1978	NO DATA
		other periods 8 readings per day

Maximum Events With Speed > 22 knots			Maximum Wind Speed and Directions are presented below		
Year	Month	Date	Hour	Speed Knots	Dir. deg.
1970	AUG	01	00	30	200
1970	OCT	23	12	23	50
1971	SEP	30	03	23	90
1973	JAN	09	03	25	360
1974	FEB	24	09	25	360
1974	OCT	23	09	23	60
1976	MAR	03	09	23	90
1978	MAY	04	09	30	360
1979	APR	04	09	25	270
1980	MAY	14	00	30	130
1982	AUG	02	06	25	50
1986	SEP	27	00	24	50
1988	MAR	01	09	24	270

189

Table 4.7 Maximum Wind Speeds Rangpur

Station: RANGPUR

Data Coverage:	JAN 1970 to DEC 1988	both Months included
Data Details :	JAN 1970 to FEB 1971	DATA 00, 03, 06 & 12 H
	MAR 1991 to MAY 1971	NO DATA
	JUN 1971 to JUL 1972	DATA 00, 03, 06 12 & 18 H
	AUG 1972	NO DATA
	SEP 1972 to JAN 1977	DATA ONLY 3-5 times a day
	FEB 1977 to MAR 1977	NO DATA
	APR 1977 to OCT 1977	DATA 00, 03, 06, 12 & 18 H
	NOV 1977	NO DATA
	DEC 1977 to JAN 1979	VARIABLE DATA COVERAGE
	JUL 1981 to SEP 1981	NO DATA

Maximum Events With Speed > 22 knots

Maximum Wind Speed and Directions are presented below

Year	Month	Date	Hour	Speed Knots	Dir. deg.
1979	FEB	21	09	25	360
1979	MAR	07	09	25	340
1979	AUG	16	00	25	90
1982	AUG	03	09	24	50
1985	APR	11	06	25	270
1988	JUN	11	06	26	130
1988	JUN	23	21	26	50
1988	JUN	24	00	26	50
1988	JUN	24	03	26	70

200

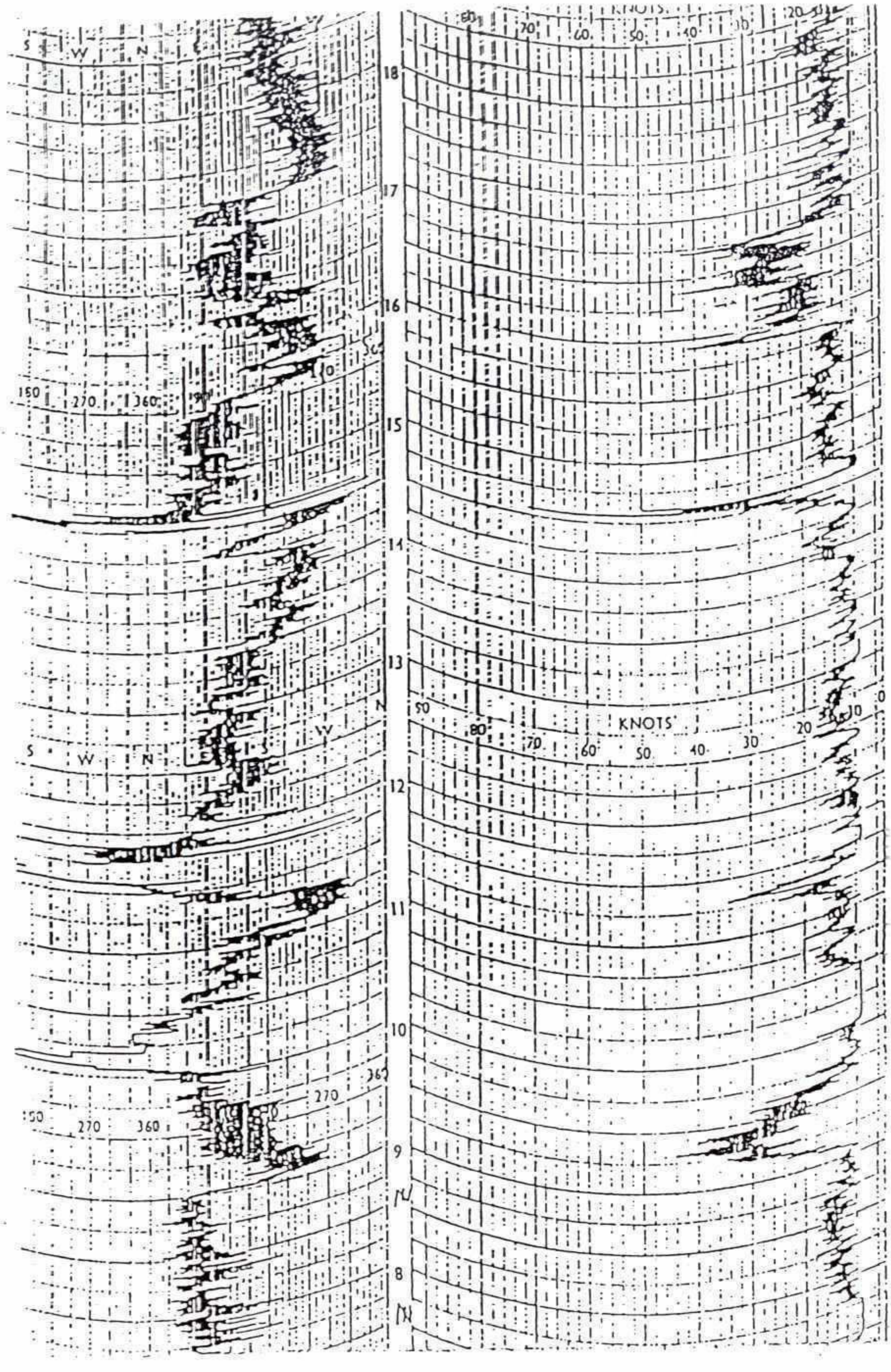
Table 4.8 Maximum Wind Speeds Mymensingh

Station: MYMENSINGH

Data Coverage: JAN 1970 to DEC 1988

Data Details :	
JAN 1970 to MAR 1970	DATA 00, 03, 06 & 12 H
MAR 1970 to MAY 1971	DATA 00, 03, 06 & 12 H
MAY 1971 to DEC 1991	NO DATA
JAN 1972 to FEB 1972	DATA 00, 03, 06 & 12 H
MAR 1972	NO DATA
APR 1972 to FEB 1972	DATA 00, 03, 06 & 12 H
MAR 1973	NO DATA
APR 1973 to AUG 1973	DATA 00, 03, 06 & 12 H
SEP 1973 to DEC 1973	NO DATA
JAN 1974 to APR 1974	DATA 00, 03, 06 & 12 H
MAY 1974 to JUN 1974	NO DATA
JUL 1974 to OCT 1974	DATA 00, 03, 06 & 12 H
OCT 1974 to JAN 1975	NO DATA
FEB 1975 to APR 1975	DATA 00, 03, 06 & 12 H
MAY 1975 to DEC 1977	SPORADIC DATA coverage
JAN 1977 to MAR 1978	DATA 00, 03, 06, 09 & 12 H
APR 1978 to JAN 1980	8 Reading per day
FEB 1980 to APR 1983	DATA 03, 06, 09 & 12 H
MAY 1983 to JAN 1987	DATA 00, 03, 06, 09 & 12
FEB 1987 to DEC 1988	8 Readings per day

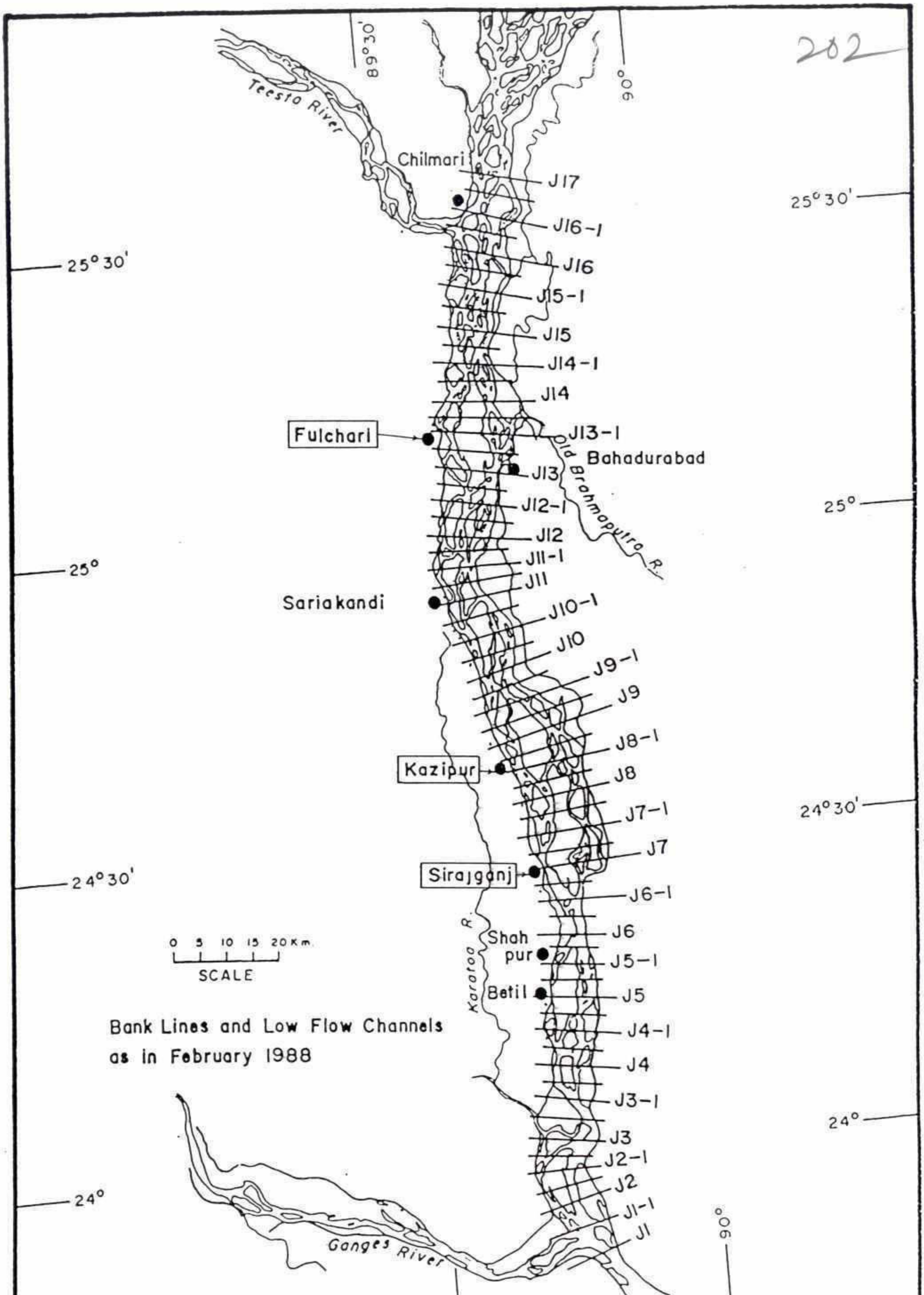
Maximum Events With Speed > 22 knots			Maximum Wind Speed and Directions are presented below		
Year	Month	Date	Hour	Speed Knots	Dir. deg.
1977	MAY	13	06	24	90
1982	AUG	02	03	25	90
1982	AUG	02	09	28	50
1982	AUG	02	12	30	50
1986	APR	19	12	23	90
1986	SEP	27	03	25	90
1986	SEP	27	06	25	90
1986	SEP	27	09	25	90
1988	SEP	27	12	24	90



① ② ③ EXTRACT FROM ANEMOGRAM FOR MARCH 25th, 1985

EXAMPLE OF ANEMOMETER RECORDING
(From Jamuna Bridge Study Report)

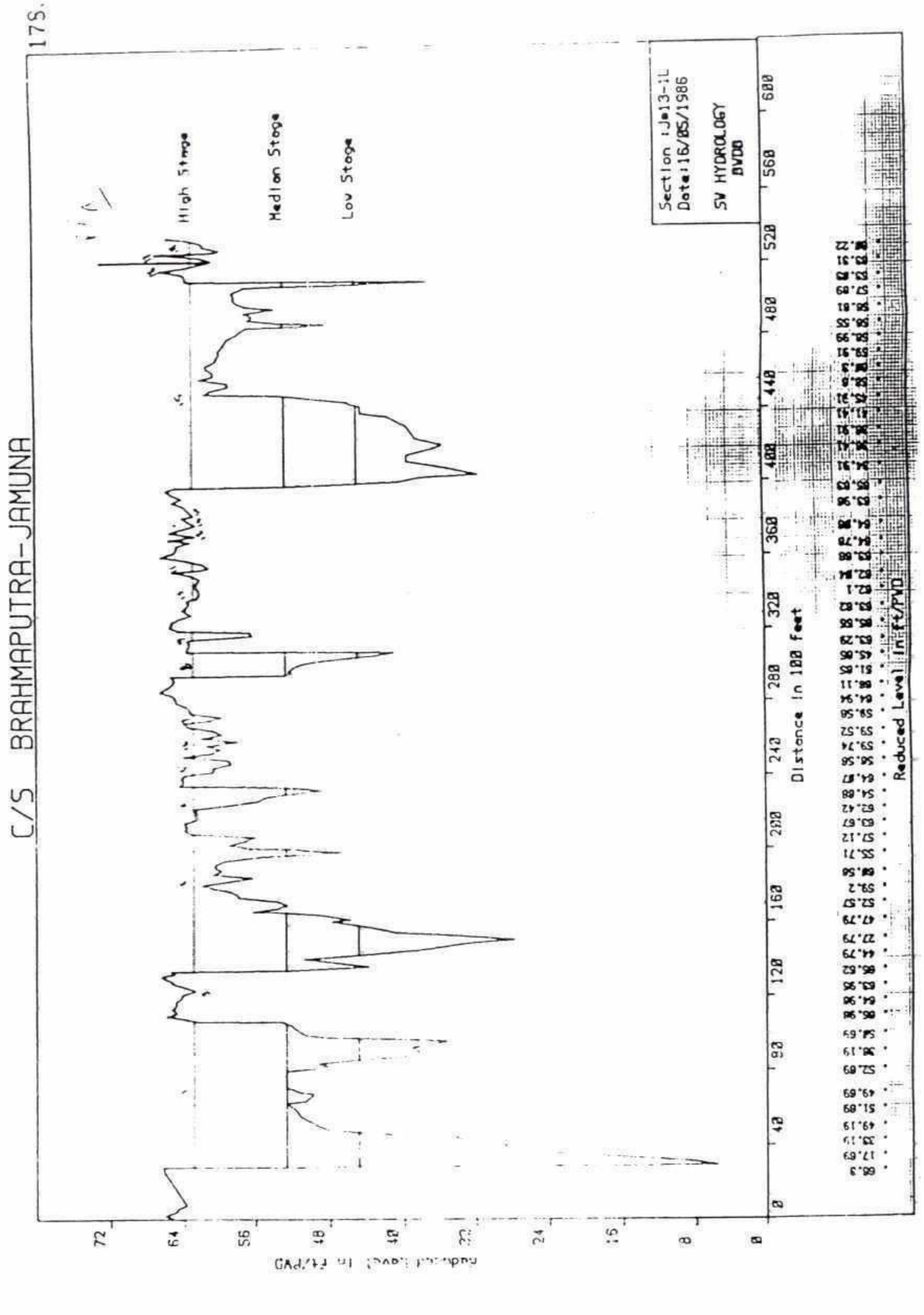
FIGURE : 4·1



POSITION OF CROSS-SECTIONS

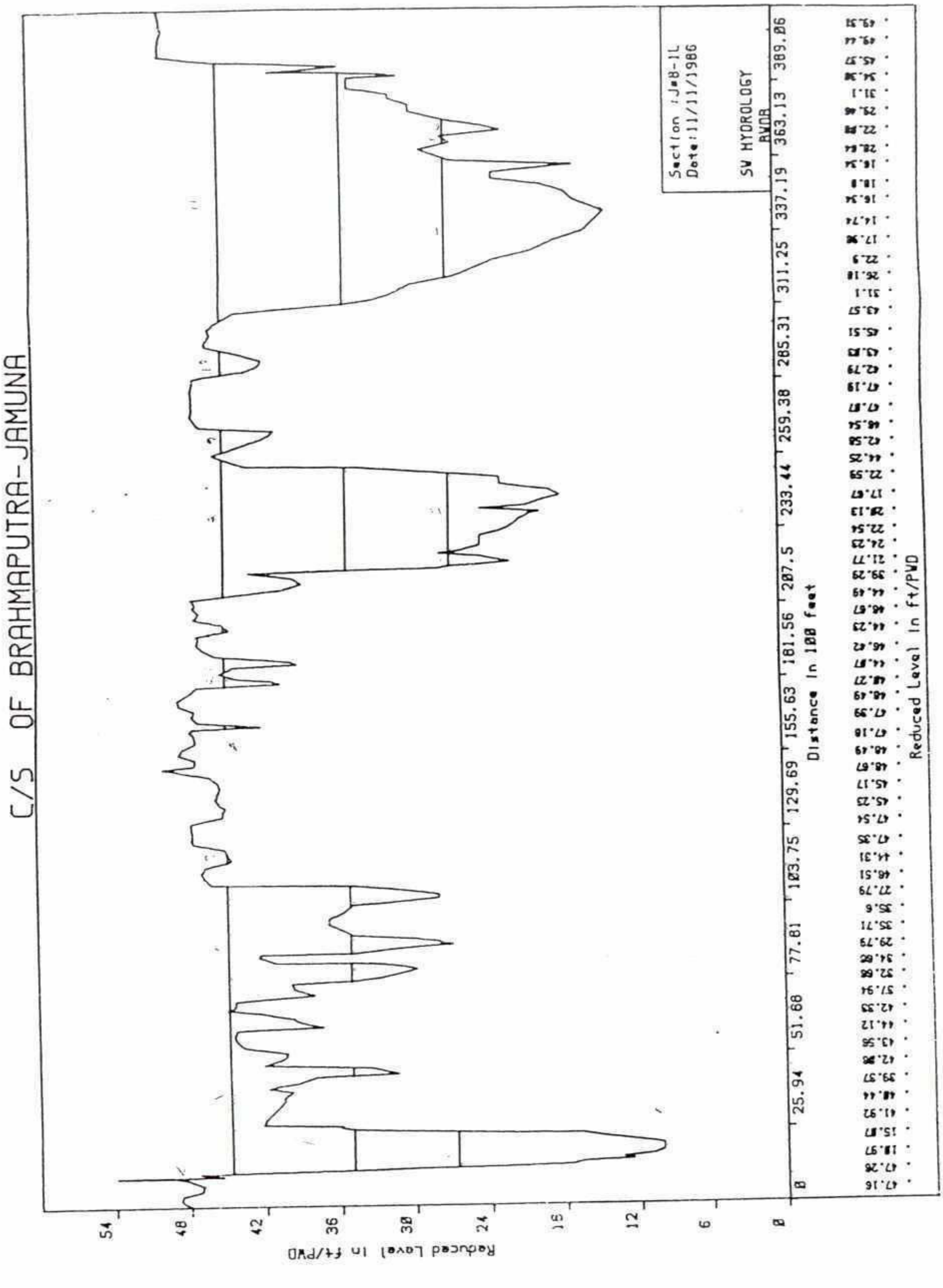
FIGURE : 4·2

203



CROSS-SECTIONS J 13-1 AT FULCHARI

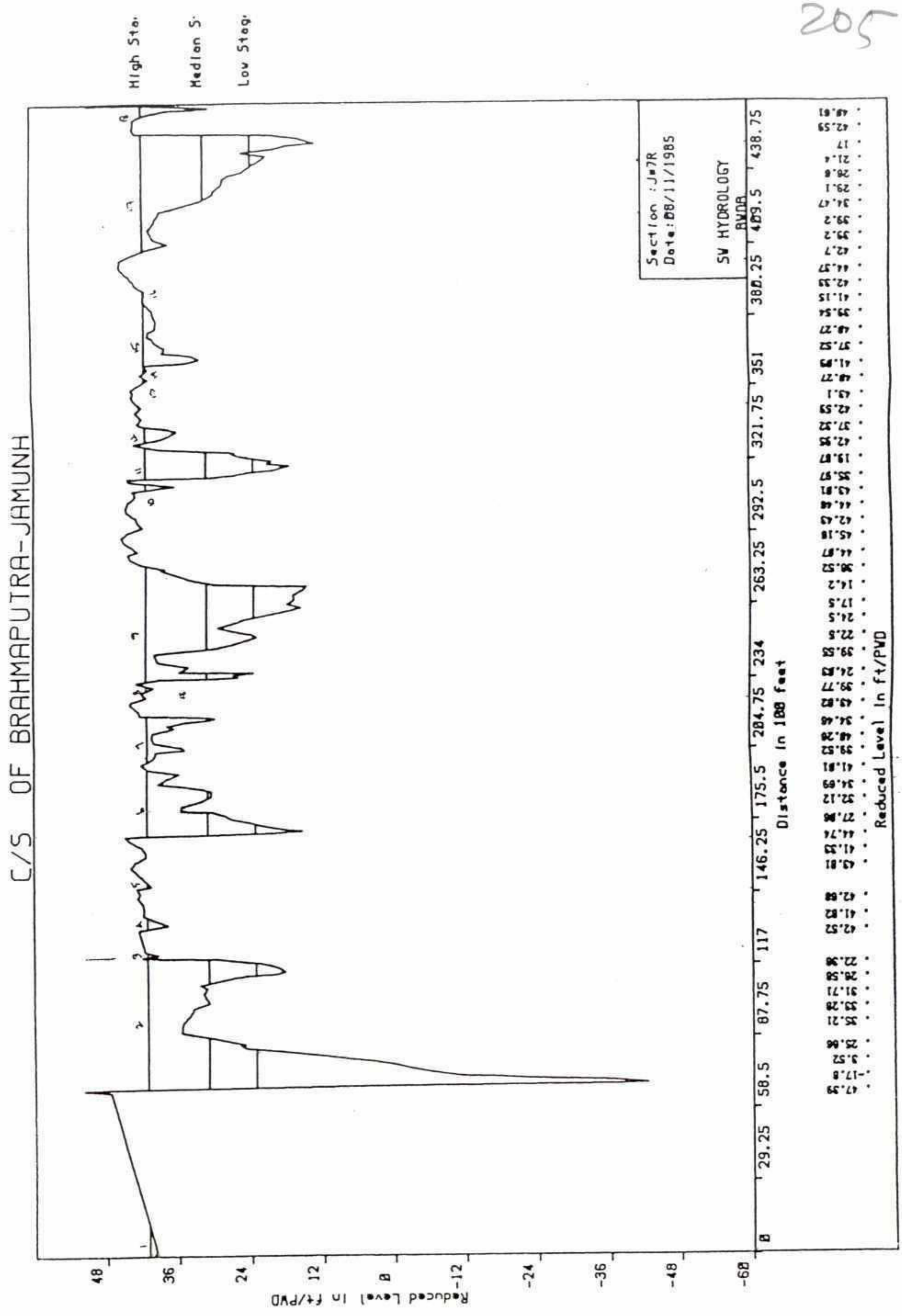
FIGURE : 4·3



CROSS-SECTIONS J 8-1 AT KAZIPUR

FIGURE : 4·4

205



CROSS-SECTIONS J 7 AT SIRAJGANJ

FIGURE : 4·5

206

APPENDIX I

DESIGN OF RIVER TRAINING WORKS - GEOTECHNICAL ASPECTS

BRAHMAPUTRA RIVER TRAINING SCHEME

DESIGN OF RIVER TRAINING WORKS

Geotechnical Aspects

1 GENERAL

The stability of the proposed revetment works has been analysed by limit equilibrium methods for both static and seismic loading. The works at Mathurapara represent the most extreme conditions and the proposed river section there has been used for the stability analyses.

Modes of failure not analysed are:

- liquefaction following an earthquake. The susceptibility of the embankment soils to liquefaction is discussed in the report on Geotechnical Aspects of Design prepared by the Geotechnical Specialist during his visit in June 1991.

- overtopping of embankments and seepage through them from the river. Provided the crest of the embankment is resistant to erosion and the crest is wide enough, overtopping will not cause severe damage. Protection to the inland toe of the embankment will prevent seepage causing any damage.

2 METHODS OF ANALYSIS

Both static and seismic loadings have been analysed using the firm's in-house computer program, SLIP5. SLIP5 was developed some 20 years ago and has been used very extensively in the firm's geotechnical practice since that date. The program has been validated for static loadings and, in addition, results for one run were compared with results from another commercially available program; it gave the same results.

The program is based on the Bishop simplified method (Bishop, 1954) for circular arc slip surfaces. It deals with submerged slopes by taking submerged weights below the external water level.

Seismic loading has been modelled by a pseudo-static option available on SLIP5. In this, a horizontal force is applied to the centre of gravity of each slice. While this element of the program has not been validated, we have not come across another program which has a validated pseudo-static option. It is generally accepted that pseudo-static analyses do not model well the forces that are applied during an earthquake. Comparative studies reported by Seed (Seed, 1979) show that the results of such analyses are conservative if the full seismic coefficient is used for the horizontal earthquake force and he recommended that horizontal forces of 0.10g and 0.15g be used for earthquake magnitudes of 6.5 and 8.2 respectively, with a factor of safety of the order of 1.15. This philosophy has been adopted for the stability analyses carried out for the design of the revetment slopes.

208

3 MATERIAL PROPERTIES

The site investigations have proved the following generalised profile with depth:

- 0 to 10m below ground level: the soils vary from clays of low plasticity through silts of low plasticity to non-plastic silts and silty uniform fine sands. The thicknesses and depths of these materials vary widely within the 10m depth from section to section.

- 10 to 20m below ground level: silty uniform fine sand

- 20 to 30m below ground level: slightly silty fine sands, fine sands, medium-fine (predominantly fine) sands, highly uniform.

A detailed description of the results of the site investigation and the stratigraphy they revealed is given in the report referred to above, the report on the site investigation and the PF calculation series also prepared in June 1991. The recommended effective shear strength parameters and bulk densities for use in the stability analyses are summarised below and their derivation is given in the calculation series PF4:

Depth range (m)	Effective Cohesion (c', kPa)	Effective angle of internal friction (ϕ', degrees)	Bulk Density (kN/m^3)
Fill	0	28	19.5
0 to 10	0	28	19.5
10 to 20	0	32	19.5
20 to 30	0	33	19.5
cc blocks	0	40	20.0

It has been assumed that the fill for embankments will be taken from shallow excavations within the top ten metres of natural ground.

4 WATER LEVELS

Analyses have been carried out for the river at flood level and at low water level. In the case of the former, ground water level has been assumed at ground level. For the river at low stage, a ground water level 1m above this level has been assumed as piezometric measurements show that there is a lag of about 1m between river level and piezometric level in the less permeable material in the upper ten metres of river bank. Piezometers in the lower strata showed an almost instantaneous response in groundwater level to river level.

5 ACCEPTABLE FACTORS OF SAFETY

A discussion of acceptable factors of safety is given in the Geotechnical Report by the Geotechnical Specialist previously referred to. At that time it was thought that there would be some aggradation in the scour hollow which would mean that the full scour depth could be cobsidered as a temporary condition and lower factors of safety would be acceptable. However, hydraulic model studies indicated that there was unlikely to be significant aggradation and therefore the minimum acceptable factor cf safety for the scour slope and the minimum acceptable factor of safety for the overall slope need to be higher than was originally recommended. The minimum factors of safety for which the slope has been designed are therefore 1.5 for static loading and, generally, 1.1 for seismic loading with a coefficient of 0.15g (this aspect is further considered in the discussion of the results of the stability analyses).

6 REVETMENT SLOPE AND SCOUR EFFECTS

The thickness of bank protection has been assumed to be 850mm, ie one thickness of cc block and 350mm of brick filter sitting on a geotextile filter. The thickness of the falling apron once it has been launched and scour has reached its full extent has been assumed to be two thicknesses of cc block.

Preliminary stability analyses were carried out assuming a revetment slope of 1 vertical to 3 horizontal and scour slopes ranging between 1:1.5 and 1:1.7. These analyses gave factors of safety that were inadequate both for static and seismic loading. With the results of the hyraulic modelling tests it became clear that overall scour slopes were approximately 1:2 and that there was a trough formed in the bed about 9m from the toe of the scoured slope. A series of analyses were therefore carried out assuming a revetment slope of 1:3.5, a scour slope of 1:2 and a trough in the river bed (see Table 1). Factors of safety for the scour slope were still low for the maximum height that was likely to occur (at Mathurapara) and the depth of scour was therefore reduced by assuming that the existing bed would be lowered by dredging to the maximum feasible depth using dredging plant available in Bangladesh. This revised section was analysed and the results are given in Table 2. Finally, the effect of an embankment with a maximum height of 3.5m was included in the section and analyses were carried out for the low river stage condition which the previous analyses had shown to be the more critical. The results of these analyses are summarised in Table 3 and discussed in the following section.

4 DISCUSSION OF RESULTS

The minimum factor of safety occurs for the seismic loading condition with a 3.5m high embankment and is for the revetment slope. All other factors of safety are adequate (see Section 5 above) for the Mathurapara section with a dredged bed and for the slopes analysed. The design earthquake magnitude is 7, not 8.2, for which the pseudo-static seismic coefficient applies, and it is considered that a minimum factor of safety of 1.05 is therefore acceptable for this section.

The trough in the scoured bed, 9m beyond the toe of the scour slope, was found not to affect the lowest factor of safety for the overall slope or the scour slope.

The proposed revetments at the other locations will all have shorter lengths of revetment, shallower depths of scour and lower embankment heights (if at all). It is therefore concluded that revetment slopes designed to conform to the assumptions made for these stability analyses will be adequately stable against failure by slipping.

River Stage	Section of slope	Factor of Safety static loading	Factor of safety seismic, 0.15g
High	Revetment	[illegible]	[illegible]
	Scour	1.48	1.02
	Overall	1.87	1.30
Low	Revetment	1.79	1.15
	Scour	1.48	1.02
	Overall	1.67	1.17

TABLE 1: Stability Analyses of Mathurapara Section: Revetment Slope 1:3.5, Scour Slope 1:2

River Stage	Section of slope	Factor of Safety static loading	Factor of safety seismic, 0.15g
High	Revetment	2.36	1.40
	Scour	1.61	1.10
	Overall	2.17	1.45
Low	Revetment	1.85	1.18
	Scour	1.61	1.10
	Overall	1.91	1.31

TABLE 2: Stability Analyses of Mathurapara Section: Dredged bed, Revetment Slope 1:3.5, Scour Slope 1:2

River Stage	Section of slope	Factor of Safety static loading	Factor of safety seismic, 0.15g
High	Revetment	-	-
	Scour	-	-
	Overall	-	-
Low	Revetment	1.60	1.05
	Scour	1.61	1.10
	Overall	1.86	1.27

TABLE 3: Stability Analyses of Mathurapara Secti[illegible]: dredged bed, 3.5m high emba[illegible]ment, revetment slope 1:3.5, scour slope 1:2

APPENDIX J

ENGINEER'S ROLE IN QUALITY CONTROL ON SITE

MASTER PLAN REPORT

ANNEX 4

APPPENDIX J - THE ENGINEER'S ROLE IN QUALITY CONTROL ON SITE

1 QUALITY CONTROL

1.1 Materials

Contractor's responsibility:

The Contractor is responsible for his own quality control inspection and rectification procedures including the supply of all manpower and equipment and for testing by others where required (eg reinforcing steel, water quality, silica-alkali reaction, sulphates, cement etc.) to ensure complete compliance with the Specification.

The Engineer's responsibility is to:

- review and approve the Contractor's procedures, facilities and manpower to meet the Specification, or advise him of non-compliance if necessary;
- audit the Contractor's compliance with his Q.A. procedures by regular sample auditing techniques;
- perform random sample quality control tests;
- routinely check accuracy and calibration of the Contractor's quality control equipment including batching plant controls etc;
- continuously monitor all test results to identify trends and potential non-compliance;
- perform check tests on all new material sources;
- check and approve/reject mix designs and trial mixes.

Checking by the Engineer will not relieve the Contractor of his responsibilities under the Contract.

1.2 Line and Level

Contractor's responsibility:

The Contractor is responsible for his own quality control, inspection and rectification procedures, including the supply of all manpower, equipment and calibration.

The Engineer's responsibility is to:

- review and approve the Contractor's procedures, equipment and manpower, or advise him of non-compliance if necessary;

213

- perform random sample calibration checks on the Contractor's equipment;
- provide the Contractor with primary bench marks;
- independently check all critical alignments and levels.

Checking by the Engineer will not relieve the Contractor of his responsibilities under the Contract.

1.3 **Specified Design Tolerances**

Contractor's responsibility:

The Contractor is solely responsible for compliance with Specification.

The Engineer's responsibility is to:

- review and approve the Contractor's procedures including method statements, and manpower, or advise him of non-compliance if necessary;
- perform checks on all critical tolerances (eg reinforcement cover).

Checking by the Engineer will not relieve the Contractor of his responsibilities under the Contract.

1.4 **Dry Earthworks**

Contractor's responsibility:

The Contractor is solely responsible for compliance with Specification.

The Engineer's responsibility is to:

- review and approve the Contractor's method of working, or advise him of non-compliance if necessary;
- inspect all foundations prior to covering up and give written approval/rejection with reasons and instructions for further action;
- perform independent tests as required to confirm the adequacy of compaction.

1.5 **Dredging and Reclamation**

Contractor's Responsibility:

The Contractor is solely responsible for compliance with Specification.

The Engineer's responsibility is to:

- review and approve the Contractor's dredging and reclamation method statements, and advise him of non-compliance if necessary;

- approve the Contractor's survey methods, attend surveys and sweeps, and agree records;

- monitor sediment concentration and disposal of waste arising from dredging operations;

- monitor the placing, sampling and testing of fill material, and the removal of unsuitable material.

Approval by the Engineer of the Contractor's method statements will not relieve the Contractor of his responsibilities under the Contract.

1.6 **Revetment Works**

Contractor's responsibility:

The Contractor is solely responsible for compliance with Specification.

The Engineer' responsibility is to:

- review and approve the Contractor's method statement, and advise him of non-compliance of necessary;

- approve geotextile manufacturer(s) and testing laboratory(ies);

- approve stiffening method/material for geotextile/fascine mattress;

- approve source(s) of armour materials;

- monitor testing of armour materials;

- monitor and approve the placing and distribution of armouring above and below water.

Approval by the Engineer of the Contractor's method statement will not relieve the Contractor of his obligations under the Contract.

1.7 **Performance Testing**

Contractor's responsibility:

The Contractor is solely responsible for satisfactory performance testing.

The Engineer's responsibility is to:

- review and approve the Contractor's procedures and methods of testing, or advise him of non-compliance if necessary;

- witness all performance tests and report on compliance/non compliance.