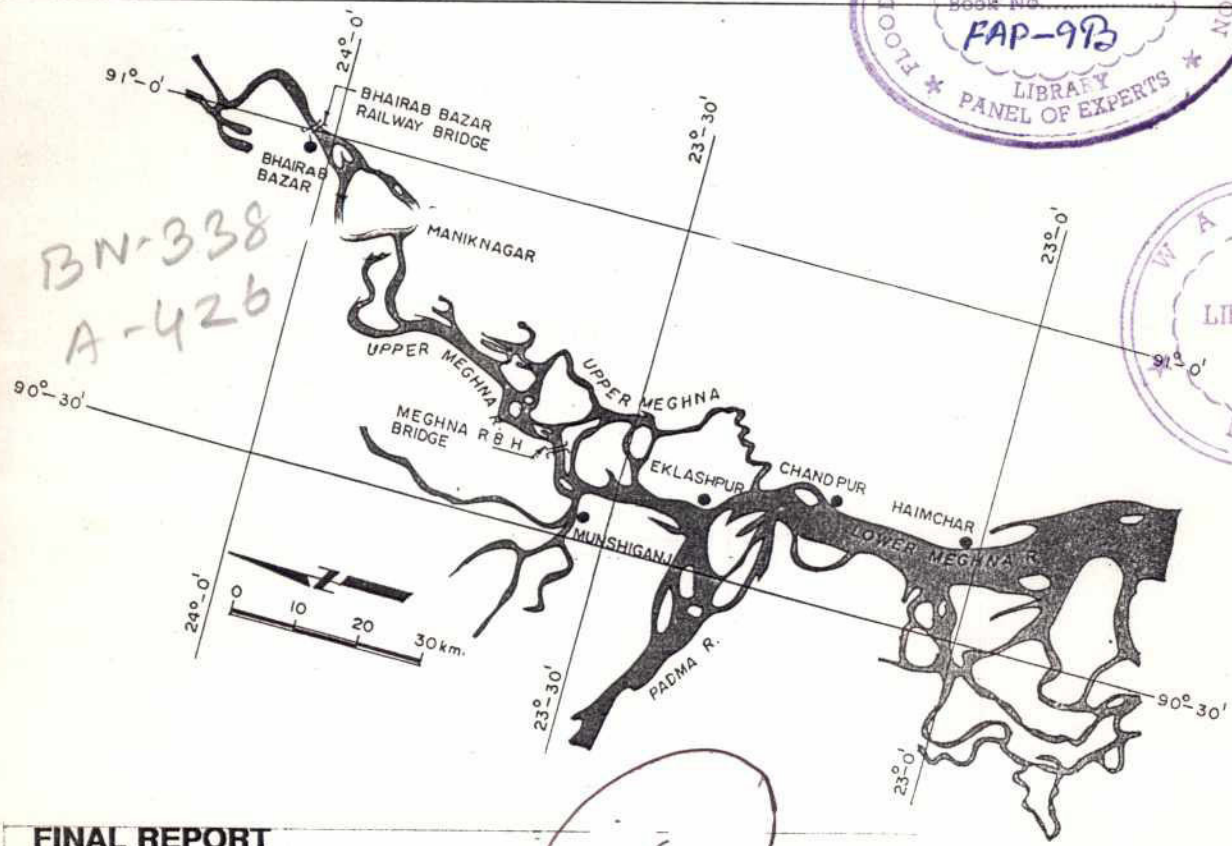
GOVERNMENT OF BANGLADESH

BANGLADESH WATER DEVELOPMENT BOARD

MEGHNA RIVER BANK PROTECTION

SHORT TERM STUDY

IDA Credit 1870 BD (Part D), March 1990



FINAL REPORT

VOLUME V

ANNEX: G **RIVER BANK PROTECTION**

February 1992

HASKONING, Royal Dutch Consulting Engineers and Architects

Delft Hydraulics

BETS, Bangladesh Engineering & Technological Services

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A-10

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HASKONING, Royal Dutch Consulting
Engineers and Architects

in association with:

DELFT HYDRAULICS
BANGLADESH ENGINEERING & TECHNOLOGICAL SERVICES LTD.

PREFACE

The Meghna, one of Bangladesh' major rivers, flows through the eastern part of Bangladesh and discharges into the Bay of Bengal.

Like other rivers in Bangladesh the Meghna erodes it banks in many points and this erosion has assumed an alarming magnitude since the severe floods of 1987 and 1988. Consequently, a number of locations requires prompt attention to prevent further damage or even events of a catastrophic nature.

This Final Report describes the surveys, studies, designs, cost estimating and economic evaluation carried out during 1990-1992 as part of the Short Term Study (FAP-9B) for Meghna Bank Protection.

The Report consists of seven volumes comprising a Main Report and nine Annexes A to I. Some Annexes are accompanied by a series of APPENDICES containing detailed information or supporting data relevant to them.

Vol I			Main Report
Vol II	Annex	A :	Hydrology
		B :	River Morphology and Geomorphology
Vol III	Annex	C :	Geotechnical Investigations
Vol IV	Annex	D :	Scale Model Studies
		E :	Mathematical Model Studies
Vol V	Annex	G :	River Bank Protection
Vol VI	Annex	F :	Economics of Protection Works
Vol VII	Annex	H :	(not used)
		I :	Environmental Impact Assessment.

INTRODUCTION TO THE PROJECT

1. **Background**

There are three major rivers in Bangladesh: the Ganges, the Brahmaputra and the Meghna. Originating form Assam in India, the Meghna River flows through the eastern part of Bangladesh and discharges into the Bay of Bengal. The Meghna River drains an area of 77.000 km^2, of which about 46.500 km^2 is located in Bangladesh. The major contributors to the river upstream of Bhairab Bazar are the Boulai, the Surma and the Kushiyara rivers, covering an area of 62.960 km^2. The Ganges joins the Brahmaputra near Aricha and thereafter takes the name of the Padma. The Padma joins the Meghna at Chandpur. The Lower Meghna River conveys the melt and rain water from the Ganges and Jamuna basins, combined in the Padma River, and from the Upper Meghna basin to the sea. The total catchment area is about 1.637.000 Km^2. Maximum flows can be as high as 1[illegible]0.000 m^3/s. The major contribution of the discharge originates from the Jamuna River (annual average 19.642 m3/s) and the Ganges River (annual average 10.874 m^3/s).

The reach of the Meghna River from Bhairab Bazar to Haimchar is about 160 km in length. Width of the river varies from 1 km to more than 10 km. The river channel is more or less well defined upstream of its confluence with the Padma and is braided in the reach downstream of Chandpur. The river is considerably deep all along and the depth ranges to 35 m in the bends. The river bed and banks consist mainly of clayey-silt which is often loosely packed and is susceptible to liquefaction at some places. Of the three major rivers, the Meghna carries relatively less sediment. The velocity of flow of the river is high during monsoon. The river banks are also subjected to heavy wave action at some points.

Like other rivers in Bangladesh, the Meghna erodes its banks in many points. Erosion at the Meghna since the severe flood of 1988 has assumed an alarming proportion at the following locations which require prompt attention.

- The Railway bridge at Bhairab Bazar;
- Bhairab Bazar Township along the right bank;
- Maniknagar; along the left bank, falling within the proposed Gumti - Phase II Project;
- Meghna R & H Bridge;
- Eklashpur (near Meghna-Dhonagoda Project);
- Chandpur Town;
- Haimchar (adjacent to Chandpur Irrigation Project);

The Dhaleswari River, a tributary of Meghna, has been eroding its right bank at Munshiganj for quite some time and has threatened the existence of Munshiganj Town.

d

2. Meghna River Bank Protection -Short term Study

The study of possible bank protection works at critical locations along the Meghna river commenced officially in September 1990 when BWDB, Bangladesh Water Development Board commissioned HASKONING. Royal Dutch Consulting Engineers and Architects in association with DELFT HYDRAULICS and BETS, Bangladesh Engineering and Technological Services, to carry out the Meghna River Protection Short Term Study. financed under Credit IDA BD-1870. Part D.

The objectives of the study are:

- to provide short term measures for protection against erosion for seven locations on the Meghna river and one location on the Dhaleswari;

- to gradually implement a coherent and phased programme of works, aiming at the control of erosion on the defined stretches of the rivers Meghna and Dhaleswari The protection of the locations indicated above should logically fit in this program

The Inception Phase started in November, 1990 with the mobilisation of the Expatriate Consultants. During the Inception Phase, the inter-action between this study and Flood Action Plan (FAP) Components was identified and maintained as far as possible.

The Meghna River Bank Protection Short Term Study, is now one of the **main components** of the Flood Action Plan for Bangladesh (FAP 9B. MEGHNA LB PROTECTION PROJECT), as included in the Review Report FPCO, December, 1990.

It has been recognised that during the Inception Phase. due to the internal and international situation during November 1990 to February 1991, delays were experienced, hampering the normal development of the activities planned. Therefore, activities in the critical path of the study were delayed (i.e, hydrometric surveys, geotechnical investigations, model investigations at RRI).

Furthermore, during the first phase of the project it became more and more clear that the inclusion of the flood season in the survey would considerably improve the designs of the protection works, the Consultants were supposed to submit at the end of the Study. Moreover, strengthening of the relation with the studies of the Bangladesh Action Plan for Flood Control (FAP) would also have a positive contribution to the outcome of this project. Therefore the BWDB instructed the Consultants to review and update the work plan taking note of the flood season of 1991 and the aforementioned studies of FAP.

As part of the Study a priority ranking was established. Accordingly, it was decided:

- to carry out a feasibility study, detailed designs and tender documents for bank protection works at the following locations:

  - Bhairab Bazar Township and Railway Bridge;
  - Munshiganj Town located on the Dhaleswari River;
  - Chandpur Town;

- to carry out a full feasibility study only for bank protection works in the following locations:

  - Eklashpur;
  - Haimchar;

- and pre-feasibility study for:

  - Meghna Roads & Highways Bridge;
  - Maniknagar, part of Gumti Phase II Project.

This Final Report submitted in accordance with the (Revised) Terms of Reference comprises all Feasibility studies carried out as well as the detailed de gns for bank protection works at the three locations mentioned above.

ABBREVIATIONS AND GLOSSARY OF TERMS

ADB	Asian Development Bank
BCSIR	Bangladesh Council for Scientific and Industrial Research
BBS	Bangladesh Bureau of Statistics
B/C	benefit cost ratio
BCL	Bangladesh Consultants Limited
BETS	Bangladesh Engineering and Technological Services Ltd
BH	Bore hole
BIWTA	Bangladesh Inland Water Transport Authority
BIWTC	Bangladesh Inland Water Transport Corporation
BOD	Biological Oxygen Demand
BR	Bangladesh Railway
BS	British Standards
BUET	Bangladesh University of Engineering and Technology
BWDB	Bangladesh Water Development Board
^{o}C	degree Celsius
CIF	Cost, insurance and freight
CPT	Cone Penetration Test
Crore	10,000,000
DH	Delft Hydraulics (Netherlands)
Dollar (US)	taken at an exchange rate of Tk.36 for the Study
EIA	environmental impact assessment
EIRR	economic internal rater of return
FAO	Food and Agricultural Organization (United Nations)
FAP	Flood Action Plan
F/C	foreign currency
Fig(s)	figures(s)
FML	fortnightly mean water level
FPCO	Flood Plan Coordination Organization
g	acceleration due to gravity
GL	ground level
ha	hectare(s)
hr	hour(s)
IBRD	International Bank for Reconstruction and Development
IDA	International Development Association
IRR	internal rate of return
IWTA	Inland Water Transport
JICA	Japan International Cooperation Agency
kg	kilogramme(s)
km	kilometre(s)
Km^2	square kilometre(s)
km/h	kilometre per hour
kN	kilonewton

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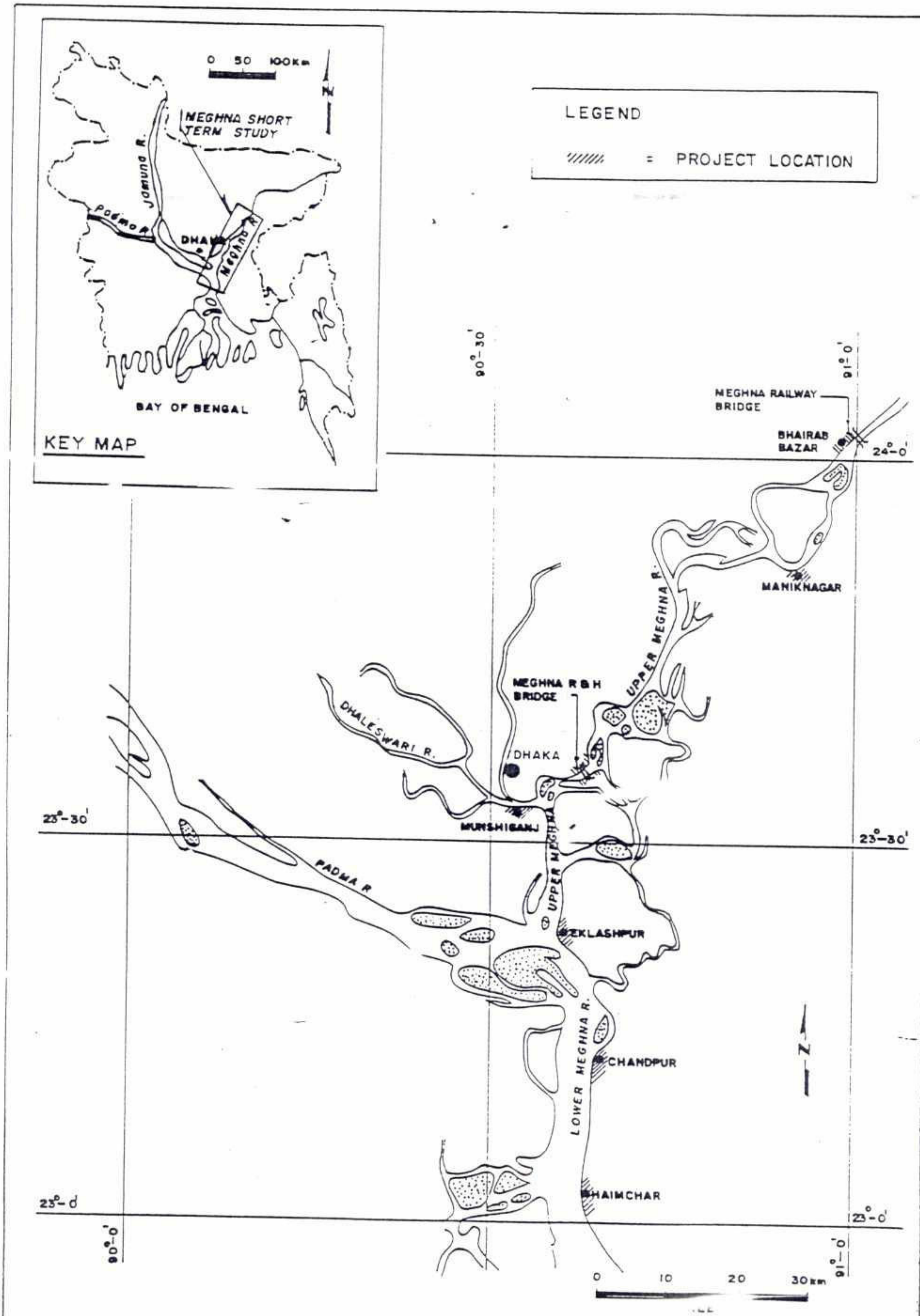
Lakh	100.000
L/C	local currency
LWL	Low water level
m	metre(s)
MAT	Manual and automatic tidal gauge
MCA	multi-criteria analysis
m/s	metre(s) per second
m^2	square metre(s)
m^3	cubic metre(s)
m^3/s	cubic metre(s) per second (cumecs)
MG	Metre Gauge
mm	millimetre(s)
MMSS	Mica schist silty sand
MN	meganewton
MPO	Master Plan Organization
MSL	mean sea level
N	Newton
NEDECO	Netherlands Engineering Consultants
NMC	natural moisture content
N-value	standard penetration test value
ODA	Overseas Development Agency
OECF	Overseas Economic Cooperation Fund
OMC	optimum moisture content
p.a	per annum
PDB	Power Development Board
PDF	Probability density function
PWD	Public Works Department (datum)
RC	reinforced concrete
RHD	Roads and Highways Department
RPT	Rendel, Palmer & Tritton Limited
RRI	River Research Insititute
RTW	river training works
s,sec	second
SHW(L)	standard high water (level
SLW(L)	standard low water (level)
SOB	Survey of Bangladesh
SPT	standard penetration test
SWMC	Surface Water Modelling Centre
sq.km	square kilometre(s)
t(tons)	metric tons
Tk	taka
TOR	Terms of Reference
US$(or$)	US dollar(s)
USCS	Unified soil classification system
WB	World Bank



INDEX MAP SHOWING PROJECT LOCATIONS FOR THE MEGHNA SHORT TERM STUDY

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ANNEX - G

RIVER BANK PROTECTION

Chapter G.1

UPPER MEGHNA

MEGHNA RIVER BANK PROTECTION

SHORT TERM STUDY

VOLUME V - ANNEX G

CHAPTER G.1 BANK PROTECTION UPPER MEGHNA SITES

CONTENTS

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LIST OF FIGURES

LIST OF TABLES

G.1 BANK PROTECTION UPPER MEGHNA SITES

G.1.1 Introduction

G.1.1.1 General

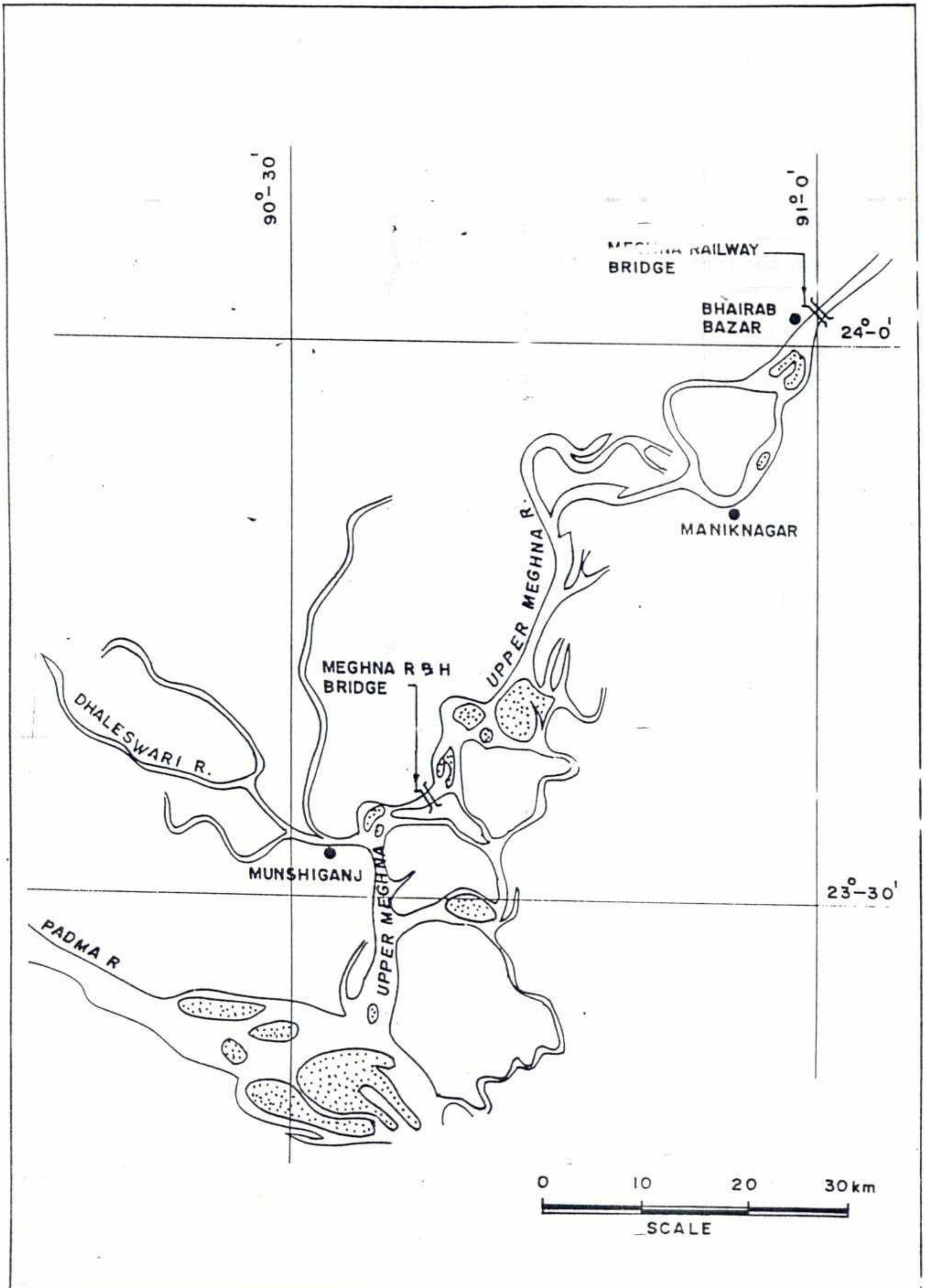
The sites which have been considered in this Short Term Study were divided into two groups: the sites along the Upper Meghna and Dhaleswari River (Bhairab Bazar, Maniknagar, Meghna Roads and Highways Bridge and Munshiganj) and the sites along the Lower Meghna (Eklashpur, Chandpur and Haimchar). In Figure G.1.1.1 a layout map of the Meghna River is shown.

Although having some stretches with a system of various channels, the Upper Meghna can be characterized as a river mainly meandering within a rather well defined high water bed, having discharges up to 20,000 m3/s. Bank protection works at each of the sites can be considered rather independently always provided that ultimately the works must fit into a more general strategic plan for the whole Upper Meghna.

The Consultants are proposing short term solutions for bank protective measures for 4 sites along the Upper Meghna River or its branches. Solutions can be either river training works, bank protection works or other protective measures.

Final designs are prepared for Bhairab Bazar Town, Bhairab Bazar Railway Bridge and Munshiganj Town whereas Pre-Feasibility level Designs will be presented for Roads and Highways Bridge (R&H Bridge) and Maniknagar.

This Annex G, Chapter G.1 deals with the design of the protection works of the Upper Meghna, whereas Chapters G.2 and G.3 will deal with works along the Lower Meghna.



LAYOUT MAP UPPER MEGHNA

FIGURE G.1.1.1

G.1.1.2 General design approach

To arrive at measures which meet the functional requirements, Consultants adopted a design approach which is presented in a schematic way in Figure G.1.1.2 and which will be discussed in subsequent sections.

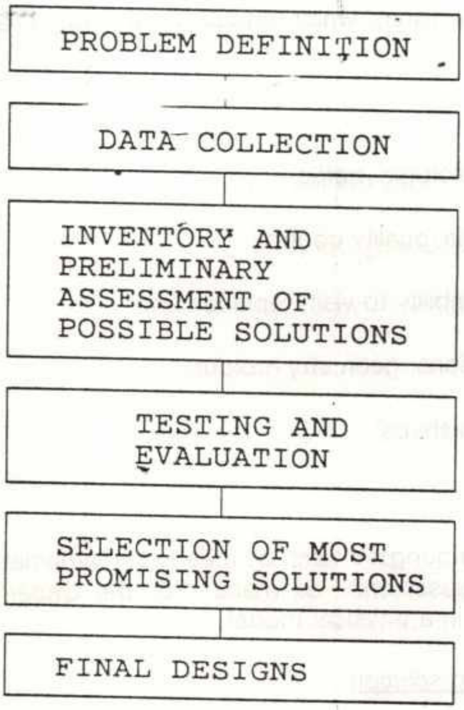


Figure G.1.1.2 Design approach

G.1.1.3 Problem definition

The problems encountered at different sites along the Upper Meghna were defined in the Inception Report and in later stages of the Study. By defining these problems functional designs can be arrived at. Problems differ from site to site and can be of a hydraulic, geo-morphologic or geotechnical nature.

G.1.1.4 Alternative solutions

Once the problem had been identified for a specific site a.. inventory of all the possible solutions was made. This implies that alternative conceptual designs were prepared. In this respect it is noticed that not only more conventional solutions but also modern design techniques were considered.

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G.1.1.5 Most promising solution

From the identified alternative solutions the most promising solution is selected. The choice between different alternatives in relation to various criteria, such as structural, social and economic aspects, is difficult to make objectively. In order to obtain a more objective selection Consultants have made use of a Multi Criteria Analysis (MCA).

Below, some possible main- and sub-criteria are listed, which should be applied in a MCA for bank protection and river training works:

- flexibility (settlements, scour);
- durability (erosion, climate, chemicals, biologic, traffic);
- construction (duration, availability, criteria/quality control);
- maintenance (monitoring, duration, possibility to visit, replacement);
- environment (pollution, secondary functions, geometry, colour);
- human factors (vandalism, recreation, mishaps).

G.1.1.6 Testing and evaluation

The solutions which are proposed were verified, amongst others, by means of mathematical models and physical models. Based on the results an assessment was made. For the Upper Meghna some alternatives for Bhairab Bazar have been tested in a physical model.

G.1.1.7 Final designs for most promising solution

For the most promising solution(s), final designs were prepared. Use was then made of a risk analysis, including a probabilistic design method.

G.1.2. ALTERNATIVE SOLUTIONS

G.1.2.1 General

In the Inception Report of this Short Term Study a priority ranking for the design of these river bank protection works has been presented. With officials of the BWDB, FPCO and the Worldbank it was agreed to deliver Final Designs (to be submitted October 1991) for Bhairab Bazar Town, Bhairab Bazar Railway Bridge and Munshiganj. For Maniknagar and Roads and Highways Bridge only Feasibility level Designs will be prepared.

In the following an appreciation of the erosion problems for sites along the Upper Meghna and the Dhaleswari River is given. Furthermore, if necessary, alternative solutions will be presented to remedy the problems.

Following the priority ranking, more attention was paid to the site of Munshiganj and Bhairab Bazar, whereas the latter is discussed in even more detail because of the local erosion problems. Physical modeling was carried out as well.

G.1.2.2 Munshiganj Town

G.1.2.2.1 General

Munshiganj Town is situated on the right bank of the Dhaleswari River, a tributary of the Meghna River, near the confluence. Munshiganj is a district headquarters and a number of important industries and cold storage facilities are located on the river banks (see Figure G.1.2.1). Erosion problems occur at both the left and the right bank of the river.

According to the Geo-morphological Study (see Annex B) it is to be expected that the actual erosion at the left bank of about 25 m/year and at the right bank of about 15 m/year will continue at this rate in future.

At the ferry ghat an area having minor erosion occurs. Active erosion was observed at the opposite site of the ferry ghat. A salt industry was eroded by the 1988 flood. The Power Development Board has relocated a transmission tower because of erosion. The bank erosion pattern indicates that the thalweg in the river bed has been shifting continuously.

Site visits of the Consultants confirm earlier reports that serious wave attack during high river stages is the most important cause of damage, especially during the early monsoon. Apparently, erosion is not due to earth slides into deeply scoured river channels. At most locations a foreshore exists having a width of 25 to 50 m or even more.

G.1.2.2.2 Alternative solutions

On the basis of the above, the following two aims can now be defined for Munshiganj:

i) prevent severe scour development in front of the foreshore;

ii) prevent erosion due to wave attack.

Bearing the above mentioned aims in mind Consultants analyzed various alternative solutions. For Munshiganj the solutions are rather simple. Three alternative solutions based on an overall revetment have been considered.

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Alternative 1 Designs according to BWDB;

Alternative 2 Protection of foreshore at existing bank ne;

Alternative 3 Protection on re-constructed embankment.

In view of the nature of the erosion problems at Munshiganj site it is not necessary to implement river training works in the very near future. For Munshiganj no physical model tests have been carried out.

In the following paragraphs the three alternatives will be discussed in some detail.

(a) Alternative 1. Designs according to BWDB

The protection works along the right bank over a length of approximately 2,000 m, proposed by the BWDB have been reviewed and improved upon. The protection consists of boulders on gunny bags and a falling apron section.

(b) Alternative 2. Protection of foreshore at existing bank line

Construction of a revetment on the existing foreshore by some filling and some excavation of areas. The area to be protected covers the entire length in front of Munshiganj. The revetment should withstand current attack and wave attack and measures should be taken at the toe to prevent failure of this part due to local scour. The latter by means of a falling apron section

(c) Alternative 3. Protection on re-constructed embankment.

This alternative, similar to alternative 2, also consists of an overall protection of the bank line at Munshiganj. The difference is, however, that the revetment is made on a re-constructed embankment which allows more working space for construction and maintains the present alignment of the main road (see Figure G.1.2.2). This embankment is made of dry sand fill, which can be obtained by dredging using a small cutter suction dredger in the river. The toe of the protection can be placed in a trench to be excavated in the foreshore, if necessary back filled with soil after construction.

Disadvantages of the first alternative are the high risk of failure due to geo-technical instability; furthermore the alternative is very expensive. Finally a combination of alternatives 2 and 3 was selected for the final design. As much as possible the existing embankment is used and by means of cut and fill a suitable slope is created on which the revetment can be made.

G.1.2.3 Maniknagar

G.1.2.3.1 General

Maniknagar is situated at a large outer bend river bank. (See Figure G.1.2.3). The active erosion starts at Shahebnagar and continues devouring the bank up to Marichakandi over a length of 16 km. There is no important township in the area but the proposed alignment for the flood embankment of Gumti Phase-II, Irrigation Project is at present within about 300 m from the river. The annual rate of erosion varies depending on its location. During moderate flood conditions such as those prevailing in 1990. The river thalweg is close to the eroding bank, the slope under water is about 1:1 over some 10 m height and the total depth varies from 25 m up to 30 m. It was reported that the river is actively eroding already for a long time and ultimately it will form a concave bank by depositing silt on the opposite bank.

Bearing in mind that this area acts as a 'hard point' it could well be that local presence of a higher clay content may have contributed to a higher erosive resistance and a relatively steep slope.

G.1.2.3.2 Alternative solutions

Upstream of Maniknagar a total erosion of the left bank of some 250 m was observed over the past 17 years. This means an annual rate of 15m. The best estimate at present is that in future this erosion will continue, because of the fact that the thalweg is presently located near the left bank. The real annual rate of erosion is difficult to estimate because of the limited reliability of the satellite images which in turn is due to small scale differences and differences in water level (see Geo-morphological Study, Annex B).

Assuming the erosion is due to geomorphological development of the river and is associated with the propagating river bend, the following alternatives solutions (see Figure G.1.2.3) can be identified:

Alternative 1 Protection works aiming at fixing the outer bend by a continuous longitudinal bank protection;

Alternative 2 Series of groynes in the outer bend;

Alternative 3 Deviating the river flow to a channel at the right bank of the river system by means of a groyne;

Alternative 4 Do nothing in the near future;

Alternative 5 Retirement of the embankment (Gumti Phase II)

In the following paragraphs these alternatives will be discussed in more detail.

(a) Alternative 1. Protection works aiming at fixing the outer bend by a continuous longitudinal bank protection

Further erosion of the outer bend can be stopped by placing a continuous protection along the left bank. The protection could consist of boulders on a geotextile and at the toe an adequate falling apron section. The length of the protection would be 5,000m from Nasirabad to Barikandi (see Figure G.1.2.3).

(b) Alternative 2. Series of groynes in the outer bend

Another possibility to stop the erosion process in the outer bend is to construct a series of 10 groynes over a stretch of 5,000m at the left bank from Nasirabad to Barikandi (see Figure G.1.2.3).

These groynes could have a length of approximately 100m each and be built up of earth with a proper slope protection.

Another possibility, falling in the same category of solutions, is applying so called sand sausages.

(c) Alternative 3. Deviating the river flow to a channel at the right bank of the river system by means of a groyne

If the discharge through the left branch could be decreased the erosion rate would also be less. For that purpose a groyne could be built just upstream of the bi-furcation upstream of Maniknagar. The length of the groyne would be 700m and consist of earth filling and a proper slope protection.

In this respect it is also possible to close the whole left branch, thus resulting in no erosion at Maniknagar at all.

(d) Alternative 4. Do nothing in the near future

Bearing in mind that the erosion rate is approximately 15 m/year while the distance between the present bank line and the alignment of the Dhonagoda Irrigation Project is 700m, it will take more than 45 years until significant damage will occur.

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As already mentioned in the Geo-morphological Study (Annex ... these figures for the rate of erosion are very inaccurate and need further studies, but nevertheless this Alternative is worthwhile to be considered.

Looking into the advantages and disadvantages of alternatives mentioned, Consultants made a pre-selection of most promising solutions. Only Alternatives 1,2 and 4 will be considered whereas the solution of deviating the flow or closing the left branch, Alternative 3, would hamper navigation in a not acceptable manner and therefore has to be rejected.

(e) Alternative 5. Retirement of the embankment

This alternative of retirement of the embankment requires more det... ...dy in long term studies or in detail in Gumti Phase II designs.

G.1.2.4 **Roads and Highways Bridge.**

G.1.2.4.1 General

The left bank abutment of the R&H Bridge is located at an eroding bend (see Figure G.1.2.4). The annual erosion reported in the last four to five years amounts to about 40 m to 50 m. A very large sand bar is slowly advancing towards the left bank. The thalweg of the river is situated near the left bank. As shown by bathymetric surveys during the construction of the Bridge, the ... bed changed substantially in 1988. The river bank erosion is due to the shifting of the main channel towards the left bank, eroding the toe of the bank slope and causing small slides.

Concentration of the river flow, propagation of the scour hole in front of the ferry ghat and constriction of the local flow profile by the construction of the bridge together cause constriction scour of the river bed under the Bridge. As a consequence thereof, a deep scour hole was formed in front of the bridge at the left side of the ferry ghat. This scour hole approaches the piers 8 and 9 of the bridge (possibly) endangering the bridges' integrity.

Upstream of the bridge the old ferry ghat acts as a hard point. Downstream there is a vortex area just before the abutment of the bridge.

When the waterlevels were receding after the flood season in 1991 the protection of the abutment collapsed at the Comilla side of the Bridge. The Contractor responsible for these protection works has taken measures to prevent further collapse of the sheetpiling and revetment. These are part of a local protection of the abutment.

G.1.2.4.2 Alternative solutions

To prevent further erosion possible types of protection can be divided into short and long term solutions. Additional[1] Short term solutions consist of protection of the piers of the bridge and the extension of the abutment protection whereas long term measures will focus on training of the river in order to prevent outflanking.

The measures being implemented at present consist of:

- bed protection around the piers by stone dumping,
- repairing bank protection of left abutment by reinforcing sheet piling, geotextile and gabion revetment.

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1 Additional in this context means in addition to the works carried o... ... main contractor for the bridge.

The Geo-morphological Study, Annex B, indicates that future development of the planform at this site implies that the upstream erosion continues at a rate of about 20m per year, while the downstream erosion may continue at the slightly smaller rate of about 10m per year.

For Meghna R & H Bridge pre-feasibility level designs will be prepared for short term measures.

In this connection Consultants have reviewed the following alternative solutions for Meghna Roads and Highways Bridge:

Alternative 1 Protection of ferry ghat and vortex area

Alternative 2 Protection of ferry ghat and vortex area, groyne of 200m length upstream of bridge

Alternative 3 Spurdike which guides the flow lines

In the following paragraphs these alternatives will be discussed in more detail.

(a) <u>Alternative 1. Protection of ferry ghat and vortex area</u>

At present the ferry ghat is acting more or less as a hard point and deflects the flow lines from the river bank which is nevertheless eroding. It is proposed to protect this 'hard point' (see Figure G.1.2.4).

(b) <u>Alternative 2. Protection of ferry ghat and vortex area and groyne of 200m length upstream of bridge</u>

The same as proposed in Alternative 1 and, to increase the deviation of the flow lines, in addition to the effect in that respect of the ferry ghat, by a groyne of 200m length upstream of the bridge (see Figure G.1.2.4).

(c) <u>Alternative 3. Spurdike which guides the flow lines</u>

A spurdike is proposed, starting just upstream of the ferry ghat and ending in downstream direction just before the bridge. The vortex area will in that case become an inland harbour (see Figure G.1.2.4).

G.1.2.5 <u>Bhairab Bazar Town and Bhairab Bazar Railway Bridge</u>

G.1.2.5.1 <u>General</u>

The Meghna River at Bhairab Bazar in this particular stretch presents deep scour holes at the right bank upstream and downstream of the existing railway bridge (see Figure G.1 2.5). The combined effects of steep underwater slopes of 1 to 1, and seepage during receding floods contribute to bank erosion.

After the 1988 flood the Meghna River started to erode the banks and a large slide occurred on the right bank immediately upstream of the Bhairab Bazar Railway Bridge.

Recently the protection works of Bhairab Bazar Railway Bridge have been strengthened by Bangladesh Railway to prevent development of unacceptable scour depths near the piers.

G.1.2.5.2 Alternative solutions

Given the present situation, the following three aims can be defined for Bhairab Bazar Town and Bhairab Bazar Railway Bridge:

i) prevent geotechnical instability of the land areas near Bhairab Bazar Town and Ferry Ghat;

ii) prevent severe scour development in front of the bank protection of Bhairab Bazar Town which scour can initiate slides or liquefaction and subsequent instability;

iii) prevent development of scour near piers of the Railway Bridge.

Using these aims as a basis Consultants have reviewed the following alternative solutions for Bhairab Bazar Town and Bhairab Bazar Railway Bridge:

Alternative 1	Maintaining present conditions;
Alternative 2	Overall bank protection on existing bank;
Alternative 3	Overall advanced bank protection;
Alternative 4	Groyne upstream of Railway Bridge;
Alternative 5	Overall protection with series of groynes upstream of Railway Bridge;
Alternative 6	Alternative 4 with low submerged sill at the Railway Bridge and guide bund at bank slide area;
Alternative 7	Bed protection at the Railway Bridge and guide bund at land reclamation in the slide area.

In the following sections these alternatives will be discussed in more detail.

(a) Alternative 1. Maintaining present conditions

Existing situation with the bank protection in its present state. However, measures, such as monitoring and maintenance, should be taken to prevent geotechnical instability (see Figure G.1 2.5 for general layout).

(b) Alternative 2. Overall bank protection on existing bank

Protection over the full length of Bhairab Bazar Town and upstream of the Railway up to the ferry ghat (see Figure G.1 2.5 for general layout).

The maximum height of the bank protection will not be beyond the existing shore level as for the design only a bank protection will be considered and no flood embankment. Nevertheless, Consultants would like to observe that a higher level of the top of the bank protection, which will then function as a flood embankment, can result in reclamation of an area in front of Bhairab Bazar Town.

When considering an overall bank protection it is possible to incorporate the existing protection works. Localised backfilling of large irregularities is possible by dumping of gunny bags filled with soil. A disadvantage of this solution is the very restricted working space.

(c) Alternative 3. Overall advanced bank protection

Alternatively it is possible to consider a so-called 'advanced' protection by formation of a strip of reclaimed land in front of the existing town protection (see Figure G.1 2.5).

(d) Alternative 4. Groyne upstream of Railway Bridge

Construction of a groyne upstream of the ferry ghat and a bank protection at the area of the bank slide (see Figure G.1 2.5). This groyne would deviate the flow from the right bank and would result in lower flow velocities along the bank protection at Bhairab Bazar Town.

Adequate measures, such as bed protection, should be taken under the middle spans of the Railway Bridge. Two possible lengths of groynes have been considered.

The top of the groyne has been determined on the basis of the 1:100 years waterlevel, which means that it should be higher than 7.79 (m +PWD). For the top of the groyne a level of 7.85 (m +PWD) has been chosen.

It is emphasized that this alternative does not solve the problem of geotechnical instability at Bhairab Bazar Town and Bhairab Bazae Railway Bridge.

(e) Alternative 5. Overall protection with series of groynes upstream of Railway Bridge

The design for protection of the Railway Bridge, consisting of a series of groynes as proposed by DDC in December 1990 [2] has also been considered as a possible alternative (see Figure G.1.2.5 for a layout) in addition to an overall protection of Alternative 2.

(f) Alternative 6. Alternative 4 with low submerged sill at the Railway Bridge and guide bund at bank slide area

This alternative concerns implementation of a groyne upstream of the ferry ghat as mentioned in Alternative 4 and a bank protection at the area of the bank slide. Between the two first piers of the bridge a low submerged sill is proposed, in order to reduce velocities along the right bank downstream of the bridge. The latter would decrease the erosion process along the revetments. Downstream of the sill erosion is to be expected due to extra turbulence. The sill would consist of boulders and a protection of, for instance, sack gabions.

Between the remaining piers a bed protection consisting of boulders on a filter layer should be added to prevent the development of excessive scour induced by the increased flow velocities due to presence of the sill.

Furthermore a simple guide bund is proposed at the area of the bank slide near the ferry ghat. An additional advantage of this solution is an extra reclaimed area. Attention should be paid to the drainage structures.

Similar to Alternative 4, a possible consequence of this alternative, viz. the sill, can be an increase of flow velocities along the left bank downstream of the Railway Bridge. Due attention should be paid to this matter.

The submerged sill is located between the piers 9 and 10 and piers 10 and 11 of the Bhairab Bazar Railway Bridge (see Figure G.1 2.5). The level of the sill has been determined by taking into account the area to be protected and the velocities over the top of the sill.

On the base of 'State of the Art' design formulas the level of the sill was determined at 12.75 (m -PWD).

Again it is emphasized that this alternative does not solve the problem of geotechnical instability at Bhairab Bazar Town and Bhairab Bazar Railway Bridge.

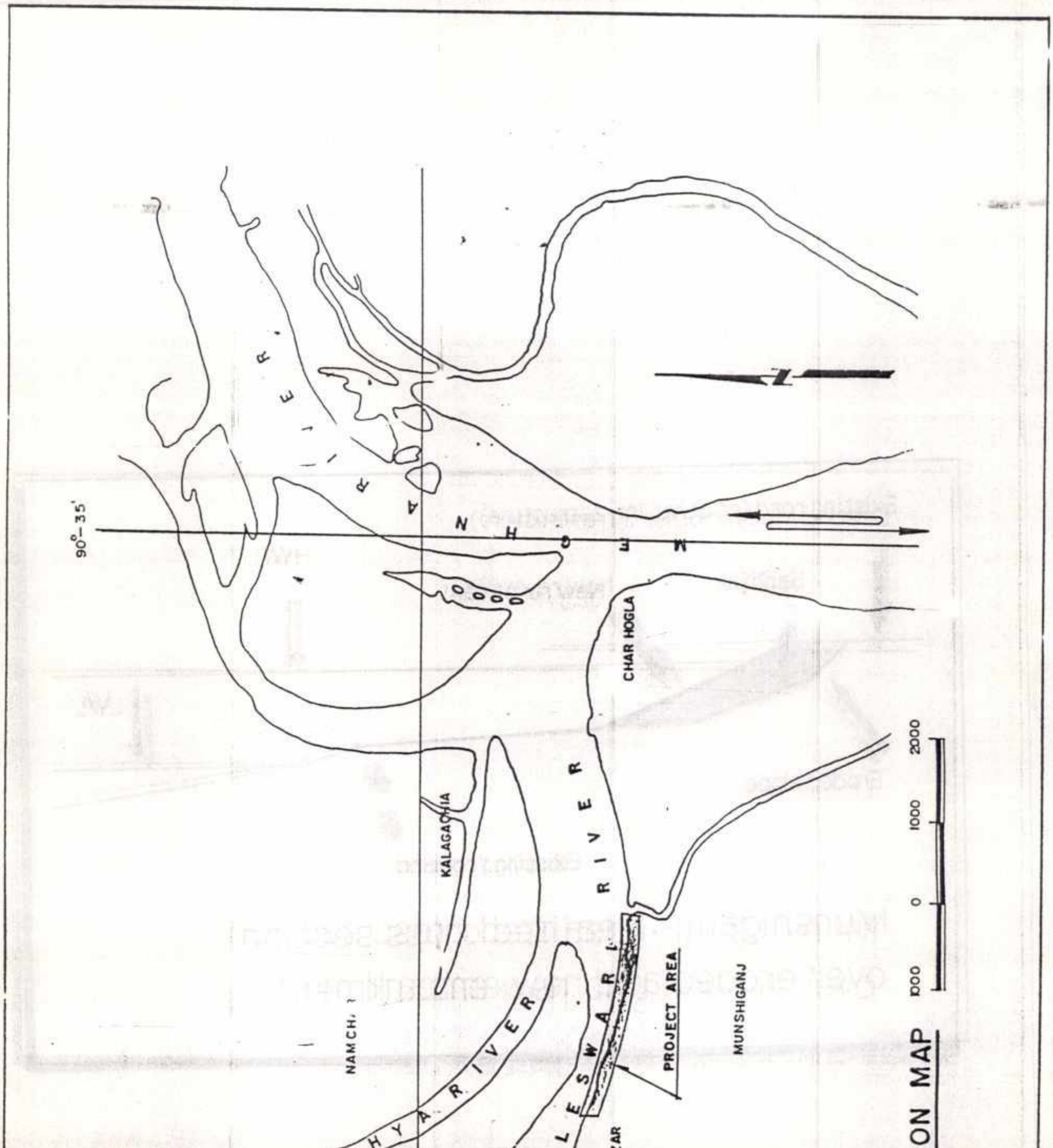
2f

(g) Alternative 7. Bed protection at the Railway Bridge and guide bund at land reclamation in the slide area

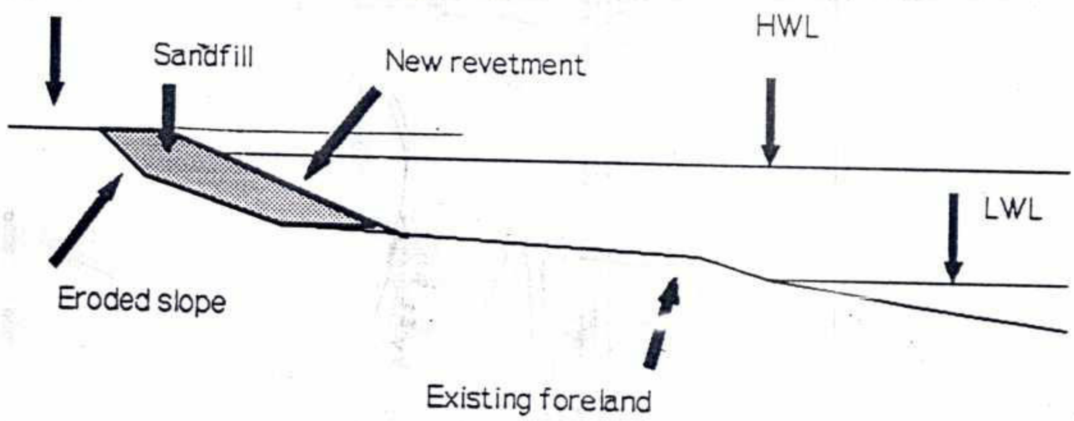
This alternative is more or less similar to Alternative 6, but now with a bed protection between the piers.

Looking into the advantages and disadvantages of alternatives mentioned, Consultants made a pre-selection of most promising solutions. The overall bank protection, Alternative 2, and the overall advanced bank protection, Alternative 3, have been selected and will be elaborated upon in the following sections.

2A



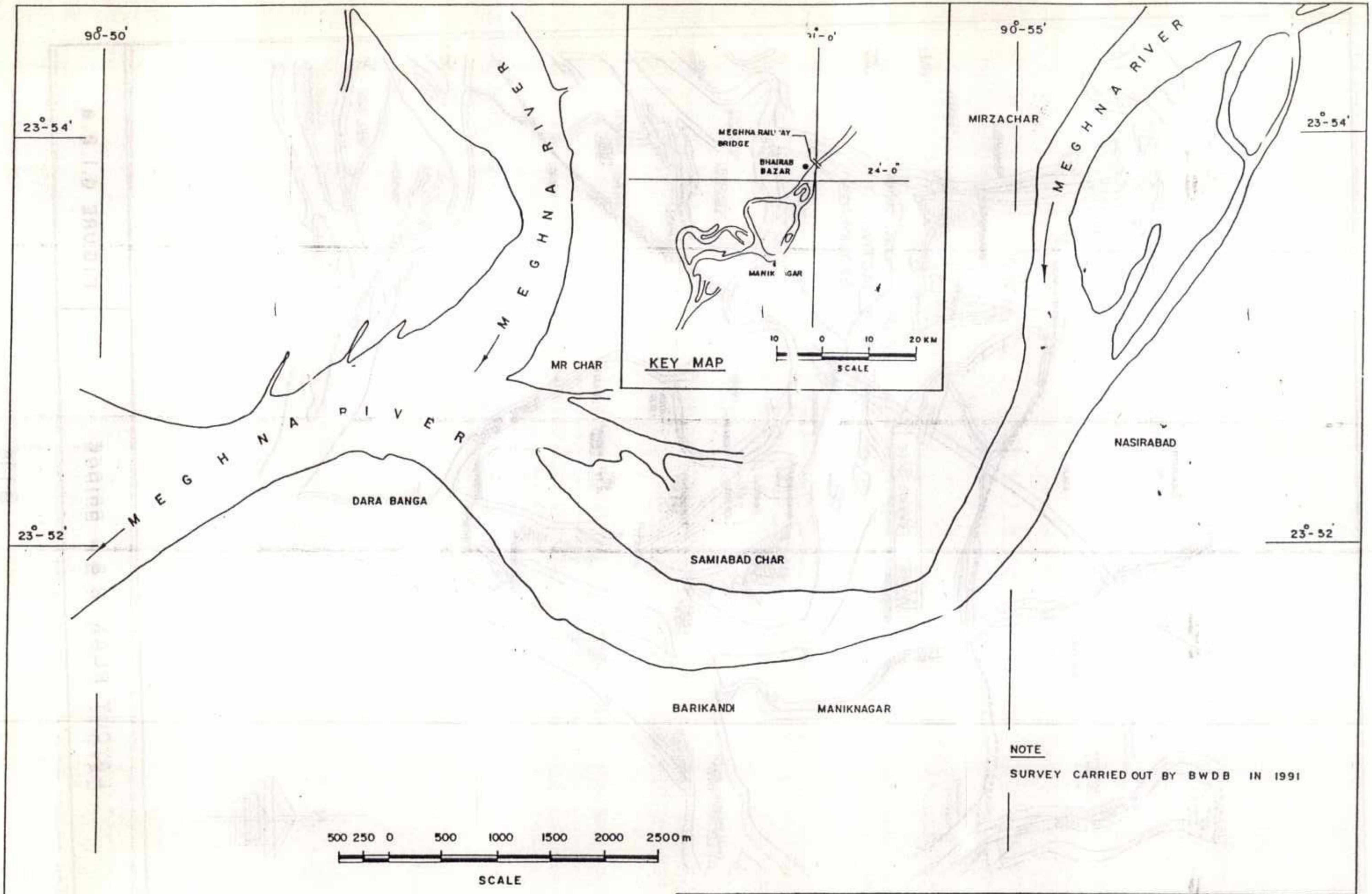
LAY OUT PLAN MUNSHIGANJ | FIGURE G.I. 2.1



Munshiganj - Idealized cross section over eroded and new embankment

CONCEPT OF NEW REVETMENT MUNSHIGANJ

FIGURE G.I. 2.2



MANIKNAGAR LAYOUT MA'

FIGURE. G.I.2.3

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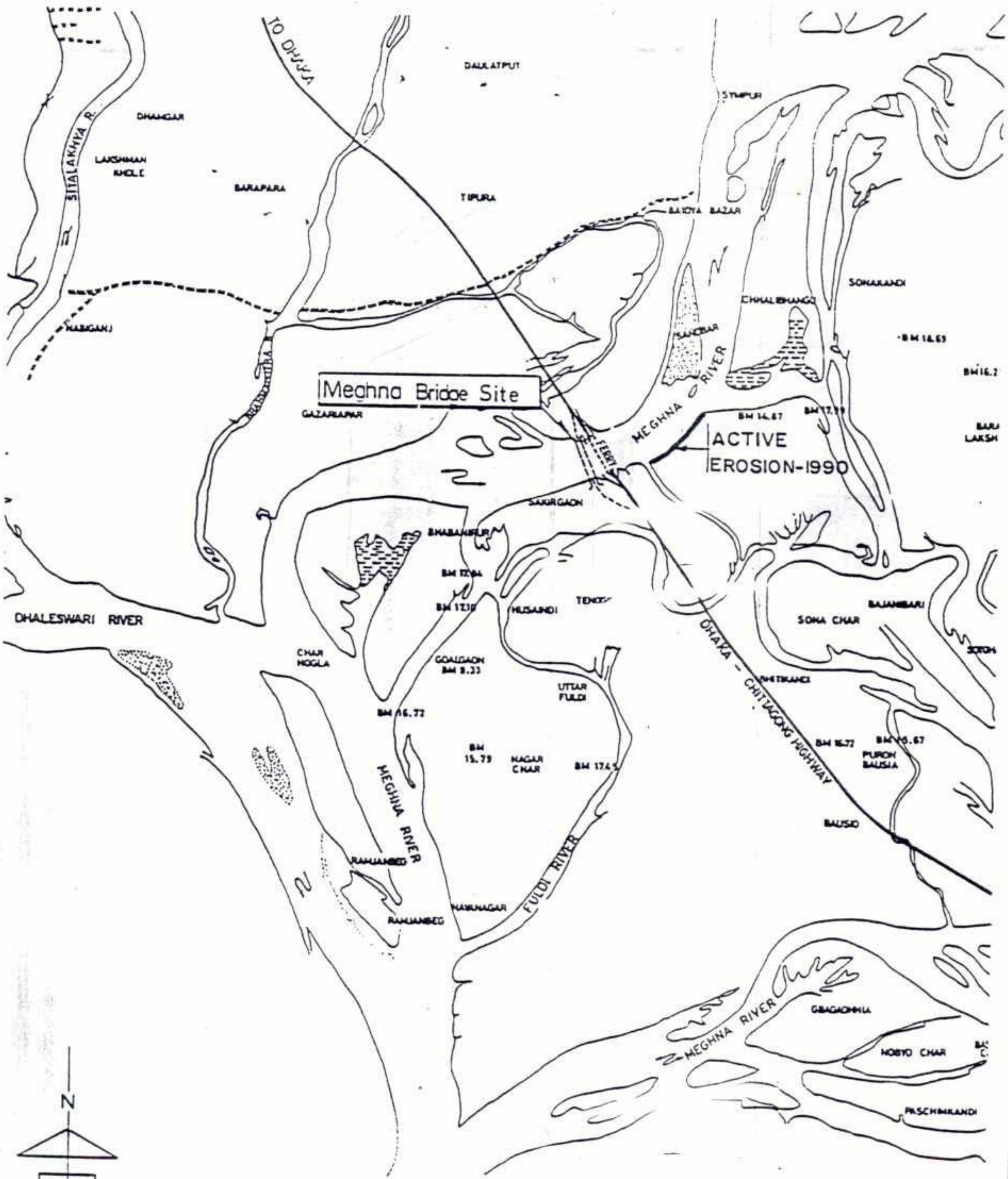

LAYOUT PLAN R & H BRIDGE

FIGURE G.1.2.4

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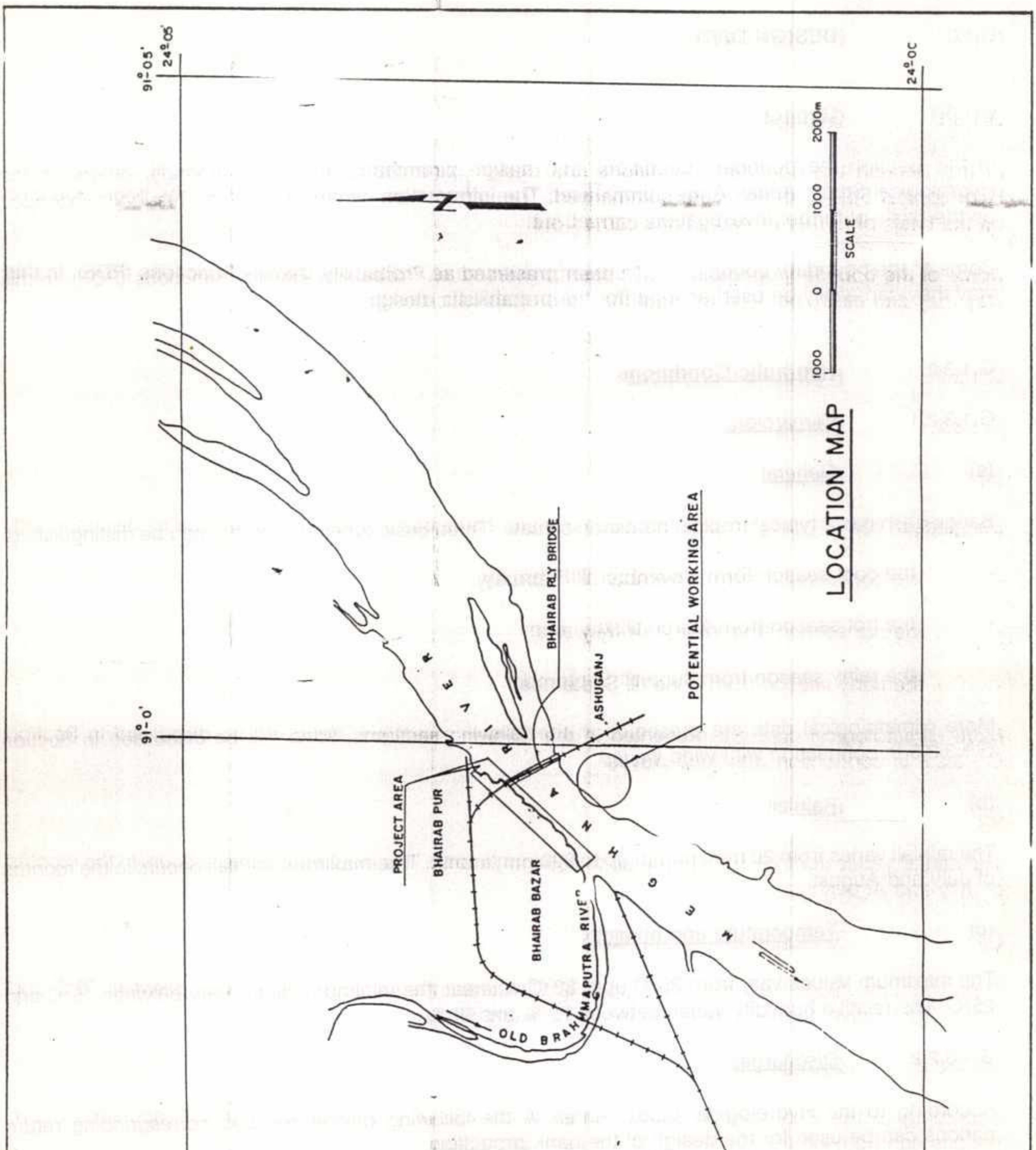

LAYOUT PLAN BHAIRAB BAZAR | FIGURE G.1.2.5

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G.1.3 DESIGN DATA

G.1.3.1 General

In this Section the boundary conditions and design parameters, as given, amongst others, in the Hydrological Study, Annex A are summarised. The information, where applicable, has been extended on the basis of further investigations carried out.

Some of the boundary conditions have been presented as Probability Density Functions (PDF). In this way they can easily be used as input for the probabilistic design.

G.1.3.2 Hydraulic Conditions

G.1.3.2.1 Climatology

(a) General

Bangladesh has a typical tropical monsoon climate. Three basic types of weather can be distinguished:

- the cool season form November till February;
- the hot season from March till May and;
- the rainy season from June till September.

More climatological data are presented in the following sections. Wind will be discussed in Section G.1.3.2.5 in connection with wind waves.

(b) Rainfall

The rainfall varies from 20 mm/month up to 900 mm/month. The maximum rainfall occurs in the months of July and August.

(c) Temperature and humidity

The maximum values vary from 25°C upto 33°C whereas the minimum values vary between 15°C and 25°C. The relative humidity varies between 75 % and 90 %.

G.1.3.2.2 Discharges

According to the Hydrological Study, Annex A the following discharges and corresponding return periods can be used for the design of the bank protection.

Table G.1.3.1 DISCHARGES UPPER MEGHNA

Return period (years)	Bhairab Bazar (m3/s)	Maniknagar (m3/s)	R&H Bridge (m3/s)
10	16,500	*)	*)
25	18,000	*)	*)
50	19,200	19,700	19,700
100	20,300	20,900	20,900

*) values not presented in Hydrological Study

For Munshiganj no discharges have been presented which can be used for design purposes. The discharge presented for Maniknagar and R&H Bridge is the combined flow in the various channels. For water levels the following discharge-stage relationship has been derived for Bhairab Bazar (see Hydrological Study, Annex A).

$$Q = 1{,}170\ (H - 1.0)^{1.49}$$

where:
H = water level (m+PWD)
Q = discharge (m^3/s)

G.1.3.2.3 Flow velocities

According to the Hydrological Study the following velocities and corresponding return periods can be used for the design of the bank protection. From the Hydrological Study only the maximum values in the profile have been considered.

Table G.1.3.2 MAXIMUM FLOW VELOCITIES UPPER MEGHNA

Return period (years)	Bhairab Bazar (m/s)	Maniknagar (m/s)	Munshiganj (m/s)	R&H Bridge (m/s)
10	1.46	*)	*)	*)
25	1.57	*)	*)	*)
50	1.67	1.26	1.74 **)	1.31
100	1.75	1.32	1.84 **)	1.36

*) values not presented in Hydrological Study, Annex A
**) values presented in Hydrological Study, Annex are very inaccurate, therefore values have been averaged.

G.1.3.2.4 Water Levels

Waterlevels have been retrieved from BWDB sources and year books and presented in the Hydrological Study. According to the Hydrological Study the following waterlevels and corresponding return periods can be used for the design of the bank protection.

Table G.1.3.3 MAXIMUM WATERLEVELS UPPER MEGHNA

Return period (years)	Bhairab Bazar Area (m +PWD)	Maniknagar (m +PWD)	Munshiganj (m +PWD)	R&H Bridge (m +PWD)
10	7.18	6.79	*)	6.00
25	7.45	7.05		6.25
50	7.62	7.22	6.41	6.42
100	7.79	7.39	6.60	6.59

*) values not presented in Hydrological Study, Annex A

Other important design levels are Standard High Water (SHW), which is the waterlevel exceeded during 18 days per year, and Standard Low Water (SLW), which is the waterlevel not exceeded during 18 days per year. They are listed in the following Table.

Table G.1.3.4 CHARACTERISTIC WATERLEVELS UPPER MEGHNA

	Bhairab Bazar	Munshiganj *)
SHW (m +PWD)	7.44	6.34
SLW (m +PWD)	1.16	0.73

*) values of R&H Bridge as presented in Hydrological Study, Annex A have been used.

G.1.3.2.5 <u>Waves</u>

Waves at the site would either be generated by wind or by ships. Data on wind waves were not available. Based on wind data from the meteorological stations Dhaka, Chandpur and Comilla predictions of the wind waves have been made. Waves generated by ships have also been considered. The data were retrieved from the Hydrological Study, Annex A and [1]. The available data comprise monthly maximum wind speed data.

For wave attack a dominant wind direction of NE has been considered with a wind velocity of 25.6 m/s and a duration of 15 minutes. For the fetch length it is considered that maximum wind velocities will occur from April till June, hence the fetch length in that case will be 2,000 m. For the waterdepth an average value of 25 m is selected. With Bretschneiders formula for wave generation a significant wave height of 0.98 m has been calculated. The period applied is 3.51 sec.

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In the following Table the wind and wave characteristics have been listed.

Table G.1.3.5 WIND AND WAVE CHARACTERISTICS UPPER MEGHNA

Return period (years)	Wind velocity (m/s)	Wave heights Hs (m)	Wave period Ts (sec)
1	15.20	0.54	2.68
10	20.40	0.76	3.12
100	25.60	0.98	3.51

G.1.3.2.6 Sediment and water characteristics

Bed samples which have been taken in the month of February 1991 were analyzed at the RRI in Faridpur. In Table G.1.3.6 some values are listed. In Figure G.1.3.1 a layout map with cross sections is shown.

Table G.1.3.6 SEDIMENT CHARACTERISTICS

	Chainage (km)	Depth (m)	D_{16} (mm)	D_{50} (mm)	D_{84} (mm)
Left	0	13.7	.09	.12	.15
	7.7	9.7	.04	.17	.21
	14	9.9	.17	.25	.29
	24	8.5	.09	.12	.19
	29	5.2	.13	.16	.22
	36	8.5	.07	.18	.23
	52	8.0	.09	.12	.21
	57	10.2	.12	.15	.19
	64	8.6	.12	.15	.20
	73	8.2	.01	.02	.09
	81	6.3	.09	.16	.22
	88	6.2	.06	.10	.15
	92	7.0	.08	.16	.22
	96	8.5	.12	.21	.26
	106	6.2	.07	.13	.18
Middle	0	11.4	.00	.02	.17
	7.7	19.3	.01	.10	.23
	14	4.1	.17	.25	.29
	24	6.2	.12	.18	.22
	29	5.3	.12	.18	.28
	36	5.8	.12	[illegible]	.21
	52	14.5	.15	.22	.33
	57	9.9	.13	.16	.28
	64	8.4	.09	.15	.21
	73	4.9	.12	.18	.21
	81	8.9	.13	.16	.28
	88	8.6	.09	[illegible]	.21
	92	9.7	.08	.10	.13
	96	11.2	.09	.13	.17
	106	12.9	.09	.13	.20
Right	0	4.2	.074	.12	.15
	7.7	27.1	.008	.06	.26
	14	6.1	.15	.2	.28
	24	7.0	.015	.04	.13
	29	15.6	.006	.05	.13
	36	3.5	.09	.12	.19
	52	11.8	.045	.18	.28
	57	6.0	.16	.24	.33
	64	9.3	.17	.25	.38
	73	7.6	.18	.24	.33
	81	20.4	.018	.07	.16
	88	11.6	.074	.25	.38
	92	7.9	.18	[illegible]	.30
	96	9.9	.01	.033	.08
	106	7.6	.007	.03	.09

The following characteristic grain size diameters have been derived from these characteristics for Bhairab Bazar:

$$\mu(D_{50}) = 0.16\,mm \qquad \sigma(D_{50}) = 0.07\,mm$$
$$\mu(D_{90}) = 0.27\,mm \qquad \sigma(D_{90}) = 0.08\,mm$$

In the same manner grain size diameters have been derived for Munshiganj:

$$\mu(D_{50}) = 0.07\,\underline{mm} \qquad \sigma(D_{50}) = 0.08\,mm$$
$$\mu(D_{90}) = 0.10\,\underline{mm} \qquad \sigma(D_{90}) = 0.08\,mm$$

G.1.3.3 **Geotechnical characteristics**

The Geotechnical Study, Annex C presents a review of all data, arriving at a specific layer classification and design parameters.

The soil of the banks has been analyzed by RRI. The following determining characteristic parameters have been selected for Bhairab Bazar:

$$\underline{D_{90}} = 0.060\,\underline{mm}$$
$$\underline{k_{soil}} = 351 * 10^{5}\,\underline{m/s}$$

whereas the same parameters for Munshiganj are:

$$\underline{D_{90}} = 0.040\,\underline{mm}$$
$$\underline{k_{soil}} = 349 * 10^{5}\,m/s$$

and for Meghna R&H Bridge:

$$\underline{D_{90}} = 0.080\,\underline{mm}$$
$$\underline{k_{soil}} = 1576 * 10^{7}\,\underline{m/s}$$

G.1.3.4 **Availability of construction materials**

G.1.3.4.1 Sand

Delivery of sand for the production of concrete will not be a problem. Often a mixture of 'local' sand and Sylhet sand is used to arrive at the proper grading.

All sand in Bangladesh has a rather high mica content, while often a high percentage of fines (diameter less than 0.063 mm) is present.

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G.1.3.4.2 Boulders

Boulders are widely available in Sylhet and, to a lesser extent, in the Rangpur area. Most of the supply is likely to come from Sylhet, where boulder collection is a seasonal activity. Experience in the past has learnt that when orders are timely placed a quantity of 100,000 m3 of boulders can be delivered to site in approximately 6 months, but not without certain problems. Such large orders do disturb the market equilibrium and force prices upward. Boulders can be used for protection of both the upper and lower parts of the slope (single boulders, gabion mattresses, sack gabions).

G.1.3.4.3 Cement

Cement is available from local sources in Bangladesh. The production of this cement is limited, however. Cement would be necessary for production of concrete blocks or cement blocks.

G.1.3.4.4 Rock

The only place were suitable rock is found at (near) surface level is in the Chittagong Hill Tracts. The existing infrastructure and security situation will probably make it difficult to obtain large quantities of rock in a short period from this area; the only way is to import it from, for instance, India or Malaysia. Rock from India would have to be transported by train. From Malaysia, from existing quarries, the required grading could easily be obtained and transported by barge to Bangladesh.

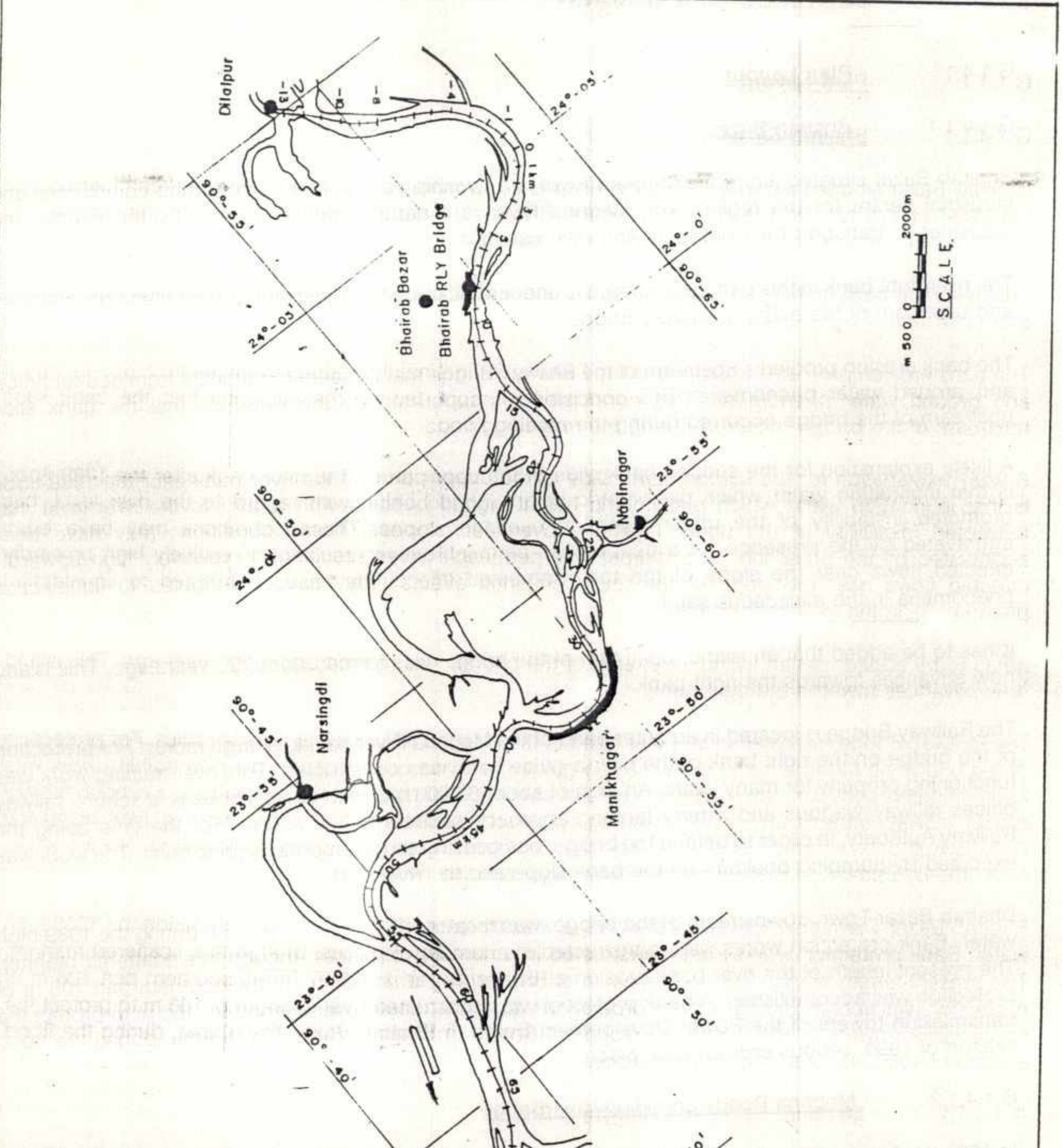
G.1.3.4.5 Bitumen

Bitumen is produced in Chittagong (East Refineries) and can be used for the production of open stone asphalt.

G.1.3.5 **Topographic and hydrographic surveys**

During the month of March 1991 detailed topographic surveys of the banks along the Meghna River at Bhairab Bazar were carried out. Similar surveys have been carried during the months of May 1991 and December 1991 for Munshiganj. For the Roads and Highways Bridge hydrographic surveys have been carried out during the month of November 1991. For the latter use has also been made of the surveys provided by the Contractor of the bridge.

Designs for the sites are based on these topographic and hydrographic surveys.



LOCATIONS WHERE BED SAMPLES ARE TAKEN

FIGURE G.1.3.1

G.1.4 DESIGN CONSIDERATIONS

G.1.4.1 Plan Layout

G.1.4.1.1 Bhairab Bazar

Bhairab Bazar situated along the Meghna River is a township as well as an important commercial and industrial centre for the region. The Meghna River is a natural water... ...d by the people and industries to transport their products and raw materials.

The river right bank in this particular stretch is under attack due to slope failure phenomena downstream and upstream of the existing Railway Bridge.

The bank erosion problems upstream of the Railway Bridge mainly seem to originate from geo-technical and ground water phenomena. This conclusion is supported by the evidence that the bank slide upstream of the bridge occurred during the receding flood.

A likely explanation for the sudden bank slide of the upper part off the river bank after the 1988 flood is that inundation water which piezometric height lagged behind with regard to the river level, has promoted instability of the upper layers in oversteep slopes. These conditions may have been aggravated by the presence of a deeper less permeable layer, resulting in relatively high upwardly directed flows near the slope of the toe. Combined effects may have contributed to liquefaction phenomena in the micaceous sand.

It has to be added that an island, upstream of the bridge, was formed about 20 years ago. This island now advances towards the right bank.

The Railway Bridge is located in an outer bend of the Meghna River having a large radius. For protection of the bridge on the right bank of the river a guide bank was constructed. This river training work was functioning properly for many years. An area of about 60,000 m2, with several houses, a school, railway offices railway wagons and a ferry landing approach suddenly ... away. For the time being the Railway Authority, in order to defend the bridge, has constructed a temporary guide bund. The work was executed by dumping boulders on the bank slope and th river bed.

Bhairab Bazar Town downstream of the bridge, was threatened by ... erosion during the 1989 high water. Bank protection works were constructed in a number of places, though in a scattered manner. The present length of the river bank erosion at Bhairab Ba ar is 628 m. In the southern part 108 m of protection was accomplished. A bank protection was constructed over a length of 183 m to protect the transmission towers of the Power Development Board. In Bhairab Bazar Town area, during the flood season of 1990, serious erosion took place.

G.1.4.1.2 Meghna Roads and Highways Bridge

In 1990 the Meghna Roads and Highways Bridge was opened officially. Some protection works at the abutments were not finalized yet at that moment. Upstream of the bridge an old ferry ghat is situated, which is now acting as a hard point and erosion of the bank line upstream of this old ferry ghat occurs.

Protection of the old ferry ghat would maintain this as a 'hard point' deviating the flow lines from the left bank. Also the upstream part of this old ferry ghat and the vortex area should then be protected. The slope protection should consist of boulders or CC blocks on a proper filter layer.

The application of sand cement elements is an economically attractive solution for both the core and slope protection of the spurdike (proposed as one of the alternatives) ... the falling apron could consist of elements of this sand cement. The construction of sand cement blocks can be carried out by local contractors. The level of the top is the same as the level of the abutment of the bridge, viz. 6.0 (m +PWD).

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G.1.4.2 **Scour depths**

G.1.4.2.1 General

Prediction of scour development in front of Bhairab Bazar has been extensively investigated in both the Physical Model tests, Annex D, and the Geo-Morphological Study, Annex B. In Section G.1.4.2.2 the results in view of design of the protection will be presented. For sites of Munshiganj and Roads and Highways Bridge, however, predictions have been made based on the results of the surveys carried out by Consultants and the Geo-morphological Study.

G.1.4.2.2 Estimate of scour Bhairab Bazar

As far as the maximum scour during the passage of a 1:100 years flood is concerned (see Geomorphological Study, Annex B), the total scour will consist of:

(a) constriction scour,

(b) bend scour,

(c) local scour.

The contributions of general scour, protrusion scour and bed form scour are assumed negligible for Bhairab Bazar. The various types of scour are discussed in more detail hereafter.

(a) Constriction scour

Constriction scour will be the result of the narrowing of the river between the Railway Bridge and the Bhairab Bazar Town site. For design purposes the following constriction scour depth has been calculated:

$$h_s = k_1 * k_2 * h_i$$

where

h_s	- constriction scour depth (m)
k_1, k_2	- coefficients derived Geo-morphological Study, Annex B
h_i	- initial waterdepth (m)
k_1	- 1.7 (-)
k_2	- 1.15 (-)
h_i	- 13.0 + 1.8 = 14.8 (m)

The calculation of the initial water depth is based on the information presented in the Geomorphological Study, Annex B. Substituting the subsequent values results in a constriction scour depth of

$$h_s = 1.7 * 1.15 * 14.8 = 29 \text{ m}$$

The presently observed water depth is 24.5 m, resulting in a scour depth referred to the initial bed level of 4.5m. This value should be corrected for an increase in waterlevel due to 1:100 flood (see Annex B). Taking into account a water level rise of about 2.3m, this induces a bed level lowering of another 2.30m. Adding this to the presently observed 17 (m -PWD) results in a bed level of approximately 19.5 (m -PWD) due to constriction scour over a length of 1,000m.

(b) Bend scour

Bhairab Bazar is located at an outer bend of the Meghna. The maximum depth resulting from scour at a natural outer bend can be expressed as a function of the average depth of the river. The outer bend scour can then be calculated with:

$$h_s = k_3 \, h_{av}$$

where:

h_s = outer bend scour below water level (m)
k_3 = factor determined by measurements (–), (in this particular case k_3 = 1.2 according to Annex B)
h_{av} = average depth (m)

For bend scour the average water depth for dominant discharge should be used. The waterlevel is approximately 5.9 (m +PWD), see Annex B, and the presently observed bed level is 17 (m -PWD) thus the average waterdepth for Bhairab Bazar during dominant discharge is:

$$h_{av} = 17 + 59 = 2290 \, m$$

Hence the water depth to be expected in an outer bend during dominant discharge is:

$$h_s = 12 * 2290 = 275 \, m$$

Thus resulting in a scour depth referred to the initial bed level of 27.5 - 22.9 = 4.6m.

The bed level taking into account the constriction scour during a 1:100 years flood is 19.5 (m -PWD) thus resulting in a bed level in the outer bend during 1:100 years flood of 19.5 + 4.6 = 24 (m -PWD). This is excluding the local scour to be discussed hereafter.

(c) Local scour

The local scour depth referred to the initial bed level in front of the slope protection can be calculated with the following formula (see Annex B):

$$\Delta h_s = \alpha \, h_0$$

where

α = 0.30 (from physical model tests)
h_0 = original water depth (m)

For the original water depth here the water depth in the constriction and in the outer bend is applied during 1:100 years flood (water level = 7.8 (m +PWD) and becomes approximately 32m (= 24 + 7.8). The local scour now becomes:

$$\Delta h_s = 03 * 32 = 10 \, m$$

The bed level near the bank protection during a 1:100 years flood is thus 24 + 10 = 34 (m -PWD). The total water depth during a 1 in 100 years flood then becomes 34 + 7.8 = 42m.

G.1.4.2.3 Estimate of scour at Munshiganj

Based on comparison of cross sections which were surveyed by the Consultants in March 1991 and in December 1991 it can be concluded that there is no significant development of scour in front of Munshiganj.

G.1.4.2.4 Estimate of scour for Roads and Highways Bridge

In view of the scour hole in front of the old ferry ghat (at present bottom level is 25 (m -PWD)) which has been developing during the last years and the preliminary results of the future geo-morphological development of the River bend, see Annex B, Consultants will adopt a scour depth of 10 m for design purposes.

G.1.4.3 **Dimensioning of the alternative structures for Bhairab Bazar**

(a) Top of bank protection on existing banks

As already mentioned before, the top of the bank protection will not be beyond the existing ground level as for the design only bank protection will be considered and no flood embankment. Bank elevations vary from 2.00 (m +PWD) upto 7.50 (m +PWD).

(b) Advanced protection

When adopting the principle of implementing an overall bank protection in front of Bhairab Bazar problems with acquisition of land for the construction of a proper berm at the top of the new revetment and adequate working space should not be underestimated. Therefore it would be advantageous to consider an advanced protection and arrive at a 'smoothly' aligned revetment.

A stable slope could be constructed by reclaiming the area by means of hydraulic fill placing in front of the existing protection and by finishing the slopes as steeply as possible. Alternatively, it is also possible to place containment bunds with a relatively small bench height under water, and fill the compartments thus formed with dredged sand.

In view of the water velocities, which will be less than 0.2 m/s during the construction season, Consultants are of opinion that the first alternative will be feasible. Slopes under water will be in the order of 1:10 up to 1:15. Therefore Consultants adopted the concept of sand pumping and trimming.

The concept of the advanced protection is shown in Figure G.1.4.1. The top level of the area which will be reclaimed is 7.80 (m +PWD) in order to cope with a 1:100 years flood.

The slope should be trimmed by dredger up to a slope of 1:3.5. After completion an appropriate protection must be placed over the sand fill.

G.1.4.4 **Flow velocities**

G.1.4.4.1 Bhairab Bazar and Munshiganj

Flow velocities for the existing situation at Munshiganj and Bhairab Bazar, which have been used for design purposes, are derived from the Hydrology Study, Annex A.

For the determination of flow velocities for several alternative solutions studied for Bhairab Bazar use has been made of the velocities which have been measured in the physical model tests with a discharge of 22,000 m3/s. In the following Table some maximum measured values are presented.

Table G.1.4.1 MAXIMUM MEASURED VELOCITIES IN PHYSICAL MODEL TESTS BHAIRAB BAZAR ALTERNATIVE WORKS

Nr.	Alternative solution	Maximum flow velocity (m/s)	Location
T9	Overall bank protection along Bhairab Bazar Town. Slope 1:2	2.05	15 m distance from top of bank protection at 0.5*waterdepth
T10	Overall bank protection along Bhairab Bazar Town. Slope 1:3	2.00	15 m distance from top of bank protection at 0.5*waterdepth
T11	Overall bank protection along Bhairab Bazar Town. Slope 1:4	1.95	15 m distance from top of bank protection at 0.5*waterdepth
T3	Existing situation and sill between piers 9 and 10 and piers 10 and 11. Sill level from 11 (m -PWD) upto 7 (m -PWD)	1.60	15 m distance from top of bank protection at 0.5*waterdepth
T4	Same situation as in 4 but with increased area of cross section	1.40	15 m distance from top of bank protection at 0.5*waterdepth
T16	Short groyne at right bank upstream of Railway Bridge.	1.70	15 m distance from top of bank protection at 0.5*waterdepth
T16	Short groyne at right bank upstream of Railway Bridge.	1.70	15 m distance from top of bank protection at 0.5*waterdepth
		2.77	In the middle of the bridge span just downstream of the bridge. Pier 7 and 8.
T17	Same as T16 but with a submerged sill	1.80	15 m distance of bank protection at 0.5*waterdepth
		3.33	In the middle of the bridge span just downstream of the bridge. Pier 7 and 8

G.1.4.4.2 Roads and Highways Bridge

Based on (i) the consideration of the maximum velocities presented in Annex A for Roads and Highways Bridge and (ii) the bed topography survey by Consultants in November 1991, a design velocity of 1.8 m/s has been selected. It is to be noted that this value differs from the one presented in Table G.1.3.2 which was a best possible estimate which could be given in Annex A.

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G.1.4.5 **Wave height**

G.1.4.5.1 Waves generated by Wind

In Section G.1.3.2.5 wind waves have been discussed.

G.1.4.5.2 Waves generated by ships

Ship traffic on the Meghna River will induce wave action, probably not higher than 0.50 m. These waves, in view of the ship traffic that can be expected at Bhairab Bazar, are of minor importance when compared to the wind induced waves. The same holds for the other sites.

So, for design purposes only wind waves have to be considered, bearing in mind that these waves are assumed to be present not only during high, but also during low waterlevels.

G.1.4.6 Environmental Aspects

During any development of a site from its natural condition there is a danger that the environment will be harmed. The Consultants are of the opinion that such a risk should be eliminated in so far as is practically possible. When the risk is above generally accepted standards then the development must be reconsidered or different solutions for the problems examined.

In the Environmental Impact Assessment, Annex I, these matters have been elaborated in more detail.

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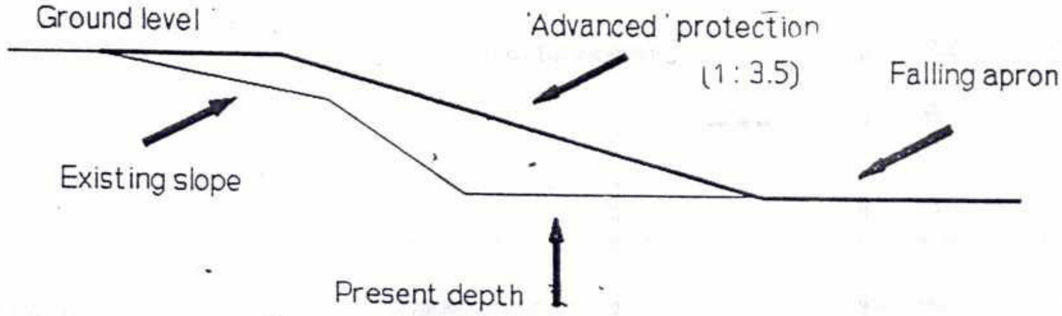
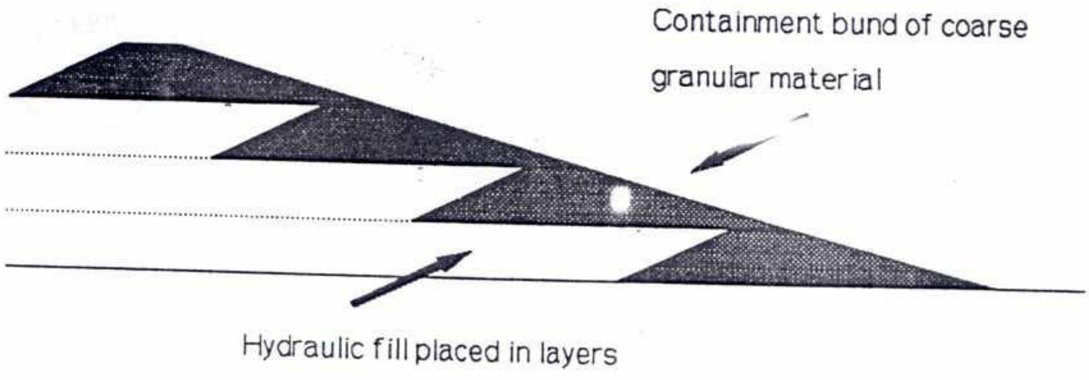


Cross section 'advanced' protection



Cross section over containment bunds and hydraulic fill

CONCEPT OF ADVANCED PROTECTION BHAIRAB BAZAR	FIGURE G.1.4.1

G.1.5 DESIGN

G.1.5.1 Geotechnical stability

In the Geotechnical Study, Annex C detailed information is presented on the various geotechnical stability aspects.

The results of extensive slope stability analyses are summarized in Table G.1.5.1.

Table G.1.5.1 SAFETY VALUES FOR 1:3.5 SLOPE ANGLES

Site	slope 1 : 3.5	
	n < 1.5	n < 1
Bhairab Bazar	1.55	1.14
Munshiganj	1.59	1.1[illegible]

A slope angle of 1:3.5 does meet the design criteria. This slope angle does also match the design requirements when evaluating micro-stability. The stability of an infinite slope will be governed by an internal friction angle $\phi' = 27°$.

G.1.5.2 Slope protection

G.1.5.2.1 Types of slope protection

For the protection of soil structures it is possible to apply "open" or "closed" revetments. In view of the differences in water levels, ground water levels behind the protection and low river water levels, a "closed" protection is not preferred. Therefore only open type structures have been considered.

In the following Consultants present an evaluation of various possibilities of slope protection methods by making use of a Multi Criteria Analysis (see Appendix G/2)

(a) Boulders or rock on geotextile

Loose dumped boulders or rock with underneath a geotextile and/or granular filter. Units should preferably be of regular form. When required the stone dumping might be followed by grouting operations.

Rock has a better interlocking than boulders. For part of the protection rock can be a good solution, while crushed rock can be applied as an aggregate for bituminous revetments.

(b) CC-blocks

Concrete cement blocks of various shape placed on geotextile and/or granular filter. The blocks should be heavy enough to prevent theft. Compared to boulders the interlocking is higher and they are highly resistant to current attack; at least two times more than boulders or rock. CC-blocks can easily be made in Bangladesh though quality control shall be an important feature.

(c) Concrete block mattresses

Block mattresses or block-mats are in general somewhat more stable than loosely placed blocks. A concrete blocks mattress consists of two components, the geotextile and the concrete blocks, which will have to function as one hydraulic structure in the final situation. The major question is whether or not the geotextile and the concrete blocks should be connected, and if so, how "tight" should this connection be. The following methods are possible:

- concrete blocks connected to the geotextile;
- concrete blocks inter-connected;
- combination of connection methods.

Practical blocks dimensions vary between 0.10m and 0.25m.

(e) Bound or grouted aggregates

Loosely packed materials bounded by cement or bitumen. The grouting mixtures can be either on bituminous or on cement-concrete basis, the former allowing for more flexibility of the grouted layer with less risk of cracking. With these relatively costly mixtures permeable as well as impermeable layers can be made. The following materials can be used:

- sand asphalt;
- asphalt mastic;
- asphalt grouted stone;
- dense stone asphalt;
- open stone asphalt.

When the asphalt has cooled down to ambient temperatures, it behaves like a solid mass with a high elasticity modulus under short loading times such as wave attack. As a plastic material of very high viscosity under prolonged loading times, it is able to follow subsoil settlements. It has been proven that by applying these bound or grouted aggregates maintenance of stone pitching can be reduced.

Open stone asphalt is prepared by mixing about 82% stones (20 - 40 mm) with about 18% pre-mixed sand mastic (i.e. 64% sand, 16% filler and 20% bitumen 80/100), giving a material in which the stones are fixed firmly and form a stable, flexible and permeable construction material (void content 25% or more, pores up to 10 mm).

(f) Wire or gabion mattresses

Wire mesh filled boxes or sacks with boulders. The size of the stone-fill must not be smaller than the size of mesh opening. The mesh opening is approximately 0.08m.

Generally, wave attack will be the determining load. The stability compared with boulders is about two times higher.

Gabions which contain, say, 5 - 10 boulders are also known as sack gabions. Handling requires no special equipment.

(g) Fabric mattresses

Geotextiles sewn together forming tubes filled with sand or cement. Special attention should be paid to the execution and the transitions which need special measures. High deformation of the mattress is possible. For slopes steeper than 1:3 the sliding criterium can be the determining one. To avoid sliding sufficient anchoring at the top is necessary.

G.1.5.2.2 Selection of type of protection

The four criteria which will be considered for selection of the type of structure are:

i	Functional requirements	Aspects such as strength against waves and currents and other functional requirements to be considered;
ii	Quality assurance	Consideration is given to the control needed to assure that the end product is what was expected. However some alternatives may be only poorly controllable, such as dumping of stones by manual labour to a depth of 50m.
iii	Maintenance	Aspects such as durability and vulnerability to internal and external damage to be evaluated;
iv	Construction	Construction includes aspects such as type of construction equipment and local or foreign labour. Here also basic materials, production and transport equipment to be considered.

The criteria are weighed as follows (see Appendix G/1 for obtaining weighing factors):

0.4*i + 0.2*ii + 0.2*iii + 0.2*iv

Two sections are considered:

- protection above water;
- protection under water.

In the following they will be discussed in more detail.

(a) Protection above water

Of the previously mentioned types of slope protection, the following alternatives will be considered for the upper part of the protection above water.

I Boulders,
II Rock,
III CC-blocks,
IV Block mattresses,
V Open stone asphalt on geotextile.

In the following the scores 1 to 5 are explained:

1	=	satisfies requirements insufficiently at all times;
2	=	satisfies requirements only under ideal circumstances;
3	=	satisfies requirements in most cases;
4	=	satisfies requirements in nearly all cases;
5	=	fully satisfies requirements.

Table G.1.5.2 MULTI CRITERIA ANALYSIS UPPER PART OF THE PROTECTION

		I Boulders	II Rock	III CC-blocks	IV Block mattresses	V Open stone asphalt
i	Functional requirements	0.8	0.8	1.5	2.0	2.0
ii	Quality assurance	0.6	0.6	0.8	0.2	0.4
iii	Maintenance	0.4	0.4	0.4	0.8	0.8
iv	Construction	0.6	0.6	0.6	0.6	0.6
TOTAL		2.40	2.40	3.30	[illegible]	3.80
Ranking		4	4	3	2	1

In the following criteria for selection are given.

(b) Functional requirements

Open stone asphalt and block mattresses are best suited to withstand the waves. CC-blocks have also a good resistance to waves. Next in order are the boulders followed by rock. The latter is not very effective compared to the other types of revetment mentioned.

(c) Quality assurance

For the upper part it will be easy to place simple elements such as boulders and CC-blocks. Block mattresses require more sophisticated methods.

(d) Maintenance

Maintenance is more or less the same for all types of revetments. However, some differentiation can be made. Block mattresses are favoured because elements are connected and can not be easily taken away. The latter is also true for grouted elements. In order of suitability considering maintenance, CC-blocks and boulders can be mentioned.

(e) Construction

Construction and or placing of boulders, gabions and CC-blocks is quite easy and could be executed by local (sub)-contractors. For the upper part for both block mattresses and open stone asphalt, foreign input is required.

(f) protection under water

The following alternatives for protection will be considered for the under water protection.

I Boulders,
II Rock,
III CC-blocks,
IV Block mattresses,
V Grouted elements.

In the following Table again the scores 1 to 5 are listed for the various alternatives.

Table G.1.5.3 MULTI CRITERIA ANALYSIS UNDER WATER PROTECTION

		I Boulders	II Rock	III CC-blocks	IV Block mattresses	V Grouted elements
i	Functional requirements	4	5	2	2	1
ii	Quality assurance	3	2	2	5	4
iii	Maintenance	4	3	2	2	5
iv	Construction	5	4	3	1	2
TOTALS		4.00	3.80	2.20	2.40	2.60
Ranking		1	2	5	4	3

In the following criteria for selection are given.

(g) Functional requirements

Boulders and (more or less equivalent rock) are most suitable for the under water protection in relation to their function. Because of their rounded shape, boulders easily start rolling on a slope if compared to rock or CC-blocks. The latter have both a higher degree of interlocking. Grouted elements and block mattresses could also be applied but are not really required.

(h) Quality assurance

Mattresses will be most easy to construct and to be monitored under water. The accuracy of placing and monitoring grouted elements is quite high. The differences between boulders, rock and CC-blocks are rather small and are mainly caused by the weight.

(i) Maintenance

Boulders and rock under water will not require much maintenance. Because of the strength of a block mattresses maintenance will also be low. However, connections between the elements can give problems. The maintenance of grouted materials is very low.

(j) Construction

The construction of a revetment incorporating grouted materials requires special equipment; the same holds for block mattresses. For utilizing such construction equipment foreign contractors are required. The other protection types, if placed with high accuracy require also special equipment. The cost of boulders will be the lowest. Sack gabions and CC-blocks require slightly more attention and are therefore more expensive.

(k) Results of MCA on selection of revetment

Based on the results of the MCA and considering the cost of the various types of protection Consultants propose to adopt the following:

- for the upper part, protection above water: stone asphalt on a geotextile.
- for the underwater protection: boulders on a fascine mattress.
- for the falling apron section: boulders having a proper grading without an underlying geotextile.

Dimensioning the falling apron will be dealt with in Section G.1.5.2.5.

G.1.5.2.3 Resistance against current attack

For the designs use will be made of the formula which have been developed in the Netherlands and adjusted for and applied to the Jamuna Bridge Project in Bangladesh.

Use is made of the Pilarczyk formula for the stability of cover layers under current attack [1].

$$\Delta_m D_n = \phi K_t \frac{0.035}{\psi_{cr}} \frac{K_h}{K_s} \frac{\bar{U}^2}{2g}$$

where:

Δ_m - *relative density* (-)
D_n - *dimensions of cover elements* (*m*)
ϕ - *stability factor* (-)
K_h - *depth factor* (-)
K_h - $(h/D_n)^{-0.2}$
k_r - D_n (*smooth units*) (*m*)
k_r - $2\ D_n$ (*rough units*) (*m*)
K_s - *slope factor* (-)
K_s - $((1-\sin^2(\alpha)/\sin^2)^{0.5}(\theta)$ (-)
$\bar{U}$ - *mean velocity* $\in$ *vertical near bank*(*m/s*)
g - *accelaration of gravity* (m/s^2)
K_t - *turbulence factor* (-)
ψ_{cr} - *critical shear stress* (-)
θ - *angle of internal friction* (*degrees*)
α - *angle of slope protection* (*degrees*)

The weight of the boulders can be derived from the dimensions of a stone according to:

$$D_{50} = 1.18\ D_n$$

$$M_{50} = D_n^3\ \rho_s$$

where

M_{50} - *mass of which is being exceeded by* 50 % *of the total mass of the batch of stones* (*kg*)
D_n - *nominal stone diameter* (*m*)
D_{50} - *50% value of size distribution* (*m*)

Firstly a deterministic design method was applied. In Section G.1.5.4 probabilistic calculations with the Pilarczyk formula have been been performed to improve the designs of the selected alternative.

In the following Table each of the parameters involved in the determination of the boulders size, will be discussed for the bank protection works at Bhairab Bazar.

Table G.1.5.4 VALUES TO BE USED FOR FORMULA UPPER PART (CURRENTS)

Parameter	Value	Remarks
$\overline{u}$	1.95 m/s	According to the maximum current velocities near the banks which have been measured in the physical model tests along overall bank protection for Bhairab Bazar area.
h	15	For waterdepth an average depth over the slope protection of 15m has been used.
Δ_m	1.60	For Bhairab Bazar the density of the water will be 1000 kg/m3 whereas per specification of the BWDB the specific density of the boulders is 2600 kg/m3. For the calculation of the relative density use has been made of these figures.
α	1:3.5	For geotechnical stability a slope of 1:3.5 is recommended. All the slopes which will be formed should stay at or below this value. Only small transition zones may exceed this value up to 1:3.5.
K_t	1.8	Consultants expect fairly high but not excessive turbulence in this area up to 20%. They recommend K_t = 1.8 for the deterministic calculations. Probabilistic calculations will be performed with an average value of 1.5 and a standard deviation of 0.15.
θ	35°	According to Lane's graph an angle of 35° has been used in the Pilarczyk formula for the boulders.
ϕ	1.00	Application of boulders on a mattress results in a stability factor of 1.00
ψ_{cr}	0.035	The critical Shields shear stress parameter which has been applied is 0.035 (no movement)

Substituting these values yields a characteristic D_n = 0.09m or a D_{50} = 0.11m. The theoretical thickness of the cover on the mattress, in view of the thickness of fascines, is approximately 0.30m. Assuming a thickness of $2*D_{50}$, boulders with D_{50} = 0.15m are proposed.

The final dimensioning will be done after the probabilistic calculations have been performed. For Munshiganj the same values have been used for current attack.

G.1.5.2.4 Resistance against wave attack

For the upper part of the protection, for the dimensioning of the characteristic dimensions of the revetment to withstand wave attack use is made of the following formula which includes the effect of the wave period [1].

$$\frac{H_s}{\Delta_m D} = \psi_u \frac{\phi}{\sqrt{\xi_z}}$$

where:

ψ_u - *upgrading factor* (-)
ϕ - *stability factor* (-)
H_s - *significant wave height* (*m*)
ξ_z - *w^ve parameter* (-)

$$\xi_z = \frac{\tan \alpha}{\sqrt{H_s/L_0}} = \frac{1.25\, T_z \tan\alpha}{\sqrt{H_s}}$$

L_0 - *wave lenght* (*m/s*)

T_z - *wave period* (sec)
α - *slope*($\angle$)
Δ_m - *relative density* (-)
D - *thickness of protection* (*m*)

The thickness of the open stone asphalt layer can be derived by filling in the values as given in Table G.2.5.5.

Table G.1.5.5 VALUES TO BE USED FOR FORMULA FOR OPEN STONE ASPHALT(WAVES)

Parameter	Value	Remarks
H_s	0.98 m	For wave attack a dominant wind direction of NE has been considered with a wind velocity of 25.6 m/s. For the fetch length it is considered that maximum wind velocities will occur in April to June, hence the fetch length will in that case be 2,000 m. For the waterdepth an average value of 25 m is selected. With Bretschneiders formula for wave forecasting a significant wave height of 0.98 m has been calculated.
T_z	3.51 sec	With Bretschneiders formula for wave forecasting a period of 3.51 sec has been calculated
ξ_z	1.00	This parameter is set at 1.0.
Δ	1.00	This parameter is set at 1.0.
α	1:3.5	For geotechnical stability a slope of 1:3.5 is recommended. All slopes to be formed should be at this angle or less steep.
ψ_u	6.00	This parameter is set to 6.0.
ϕ	1.00	This parameter is set to 1.0.

Substituting these values in the aforementioned formula yields a minimum value for the thickness of the layer of 0.15 m.

The open stone asphalt layer is placed up to 3.00 (m +PWD) and allows the placing of the open stone asphalt under dry conditions. The level corresponds approximately with a waterlevel that is not exceeded during 50% of the time (see Hydrological Study, Annex A).

For Munshiganj the same data concerning wind velocity, direction and waterdepth have been considered as for Bhairab Bazar. For the fetch length a value of 1,950 m has been used. For practical purposes this means that the same dimensions for the thickness of the stone asphalt layer as for Bhairab Bazar can be applied for Munshiganj. The open stone asphalt is here applied up to 2.75 (m +PWD).

G.1.5.2.5 Falling apron

The design of slope protection works should anticipate on expected maximum scouring depths. In many cases these maximum depths exceed largely the present river depths. To construct the slope protection completely to the final depths, often requires extensive dredging efforts. An alternative solution could therefore be a so-called falling apron protection which has been proposed for instance for the river training works for the Jamuna Bridge Project.

The most secure method to provide future coverage of the slope of the scour hole is to provide loose granular material. The length and quantity should be sufficient to cover the entire downwardly developed slope in future. The thickness and the grading of the granular material should be such that at the end of the falling process the interlaying soil is retained by the protective layer.

In the following the design for a falling apron is described which is applied in the Jamuna Bridge Project [1].

The final slope of a falling apron is assumed to be 1:2.0. The dimensions of a falling apron can be determined as follows. The dimensions have been determined by applying the same formula as for the attack for the upper part. Replacing the slope gradient of 1:3.5 and the internal stability in that calculation into respectively 1:2 and 40 degrees results in a diameter of a stone (boulder or rock) of 0.15m. Furthermore Kt has been decreased to 0.75 because the falling apron is a continuous protection.

For the falling apron a grading of boulders/rock has to be selected that contains a sufficient quantity of rocks suitable to individually resist the current forces. Also it should contain sufficient smaller particles to warrant a proper functioning of the filter to be formed. Therefore a grading of rock of 0.05 - 0.20m has been selected, containing sufficient 'fines' for filter purposes and sufficient coarse particles to resist the current forces.

Soil can in principle be retained by a filter structure built up from granular materials with different grading. The grading closest to the soil to be retained shall be rather fine, while the grading closest to the current shall be large enough to withstand the current forces. In the falling process the filter will not be built up as nicely as necessary for a proper filter function. This can be overcome by providing more granular material than would be required for a proper filter. As a practical rule an excess quantity of sixty percent of the proper filter quantity will be sufficient. This rule is more or less equivalent to another design rule, which indicates that the total thickness of an all-in filter should be approximately five times the diameter of a single boulder/rock which can (just) withstand the current forces.

Another excess quantity is required to allow for the fact that not all material in the falling apron reaches its intended destination. Here an excess quantity of twenty five percent is recommended, particularly in the Indian literature on this subject.

All considerations given can be expressed in the following formula:

$$T_f = 5 * D_{65} * 1.25 \ (m)$$
$$Q_f = D_s * T_f \sqrt{5}$$
$$L_f = 1.25 * D_s$$

where:

T_f = thickness of falling apron (m)

Q_f = quantity required in falling apron (m^3/lin.m)

D_s = expected scour depth referred to initial bed level (m)

L_f = Lenght of falling apron (m)

It Q_f is the quantity of graded rock to be stocked in the falling apron for each m2 of slope expected to be exposed as a result of scour. The slope gradient to be expected is approximately 1:2, equivalent to the natural slope. In the following Table for three sections of Bhairab Bazar/Munshiganj the dimensions of the falling apron section are listed.

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Table G.1.5.6 DIMENSIONS OF FALLING APRON SECTION BHAIRAB BAZAR/MUNSHIGANJ

Ds (m)	Lf (m)	Qf (m3/m)
11.50	14.40	24.10

G.1.5.2.6 Filter requirements

(a) Granular filters

A granular filter between subsoil and outer layer has to meet the following requirements, related to the representative grain sizes of the subsoil Db and the filter Df:

Table G.1.5.7 GRANULAR FILTER CRITERIA

Criterium	Constraints
permeability	D15f/D15b > 4 - 5
segregation	D50f/D50b < 20 - 50
piping	D15f/D85b < 4 - 5
internal stability	D60f/D10f < 10 no migration D60f/D10f > migration

For the time being no granular filters are considered.

(b) Geotextiles

Geotextiles are more and more used as separation between layers of different composition. Both woven and non-woven geotextiles can be considered. In the following Table the requirements which are used for the review are summarized [1], [3]:

Table G.1.5.8 CRITERIA FOR GEOTEXTILE FILTERS

Type of geotextile	Sandtightness	Permeability
Woven	O90/D90b < 1	kf = 5 k_soil
Non-woven	O90/D90b < 1.8	kf = 5 k_soil

in which:

O_{90} - effective opening size of geotextile (m)
D_{90b} - characteristic size of subsoil particles (m)

k_f - permittivity geotextile (s^{-1})
k_{soil} - permeability soil (m/s)

To meet the requirements a composite geotextile is required which consists of a combination of a woven, for the strength, and a non-woven, for the sand tightness.

For soil characteristics reference is made to Section G.1.3.3.

In the following Table specifications for woven geotextiles are listed.

Table G.1.5.9 GEOTEXTILE SPECIFICATIONS WOVEN (TYPE I)

Item	Specification
Type of geotextile	100 % woven polypropylene
Effective pore size	200*10-3 < O90 < 300*10-3 m
Permittivity	> 0.1 s-1
Strength wrap and weft	70 kN/m
Weight	450 gr/m2

In the following Table specifications for non-woven geotextiles are listed.

Table G.1.5.10 GEOTEXTILE SPECIFICATIONS NON-WOVEN (TYPE II)

Item	Specification
Type of geotextile	100% non-woven
Effective pore size	O90 < 0.125 10-3 m O50 < 0.075 10-3 m
Permittivity	> .1 s-1
Strength	> 70 Kn/m
Weight	> 200 gr/m2
Grab strength	> 900 N
With on roll	> 5 m

This composite geotextile will be placed underneath the open stone asphalt layer and integrated into the fascine mattress. No further distinction is made.

G.1.5.3 **Designs for various alternative schemes at Bhairab Bazar and other locations**

G.1.5.3.1 Bhairab Bazar Town and Bhairab Bazar Railway Bridge

For Bhairab Bazar following designs have been made:

I Overall protection on existing embankment, including protection between piers (Figure G.1.5.4);

II Overall protection on advanced embankment, including protection between piers (Figures G.1.5.5 and G.1.5.6).

In Section G.1.8 a further selection will be made.

G.1.5.3.2 Munshiganj

Based on the results of the Geo-morphological Study, Annex B, and an analysis of cross sections surveyed in March 1991 and December 1991 Consultants selected a protection for Munshiganj which is a combination of Alternatives 2 and 3. As mentioned before the same type of protection can be used as selected for Bhairab Bazar, the same holds for the dimensions.

Since there is no significant scour development in front of Munshiganj, see Section G.1.4.2.3, a falling apron is not required. In the bend of the Dhaleswari, however, upstream of Munshiganj, a falling apron is required, type and dimensions are similar to those presented for Bhairab Bazar.

For Munshiganj design drawings have been made for the combination of Alternatives 2 and 3 (see Figure G.1.5.1 for a layout plan). In Figures G.1.5.2 and G.1.5.3 typical cross sections for Munshiganj have been shown.

G.1.5.3.3 Maniknagar

For the most promising solutions feasibility designs have been made. In Figure G.1.5.7 a typical cross section has been shown.

Bearing in mind that (i) the erosion rate is not high and the eroding bank line will reach the embankment of the Dhonagoda Irrigation Project only after 45 years (see also Section G.1.2.3) and (ii), the low value of the benefits (see Annex F: Economics of Protection Works) Consultants propose to do nothing in the near future at Maniknagar.

G.1.5.3.4 Roads and Highways Bridge

The effect on the flow lines of a groyne and a spurdike, as proposed in the alternatives, is hard to predict without further detailed study. Since protective measures are urgently required for Meghna Roads and Highways Bridge, Consultants selected, in view of:

(i) the more favourable cost, (Section G.1.7), and

(ii) the future geo-morphological development, (see Annex B);

the alternative of the spurdike consisting of sand cement stone.

The core body of the spurdike will consist of sand cement blocks which' size will be varying between 0.10m and 0.30m.

The slope protection will consist of CC-blocks with D_{50}=0.25m and a thickness of 0.50m. This slope protection will be stable for the aforementioned currents and wave attack. It is noted that the relative density of sand cement blocks is significantly lower them the usual value of 1.65 which for instance is valid for boulders. Therefore dimensions are larger than would be expected if boulders or CC blocks were applied. However also for the alternative of a toplayer of CC-blocks cost estimates have been made.

The falling apron will consist of elements of sand cement blocks with D_{50}=0.25 with a considerably steep grading. The length of the falling apron is 15.50m and is based on the expected scour depth. The quantity in the falling apron is 30m3 per lin.m.

In Figure G.1.5.8 a layout has been presented and in Figure G.1.5.9 a typical cross section of the spurdike shaped guide bund has been shown.

G.1.5.4 **Risk Analysis**

G.1.5.4.1 Introduction

For the design of the bank protection works a risk analysis has been carried out. The objectives of the Risk Analysis are:

- to define an acceptable probability of failure of the bank protection;
- to identify and quantify the hazards of the bank protection;
- to integrate the design of the bank protection into other infrastructural works.

The three main elements in a risk analysis are hazard, mechanism and consequences. A risk analysis starts with an inventory of the hazards and mechanisms. A mechanism is defined as the manner in which the structure responds to a hazard as shown in the following scheme.

preparing an inventory of hazards
formulating the failure mechanism
calculating the failure probabilities
quantifying the consequences
risk = probability * consequence

A combination of hazards and mechanisms leads, with a particular probability, to failure or collapse of the structure or of its component parts.

Finally, the consequences of failure or collapse must be considered. In the event of failure of bank protection as a whole,the relevant damage characteristics, structural damage and duration of load must be estimated. The probability of failure multiplied by the damage or loss constitutes the risk. For an optimal design it is essential to weigh the risk against the cost of constructing a heavier structure.

Appendix G/5 presents in a more elaborated manner the risk analysis and the probabilistic design method.

G.1.5.4.2 Failure modes

Bank protection along the Meghna River is constructed to protect the population and the economic values against floods and shifts of the alignment of the river. Absolute safety is in principle impossible to realize. Therefore it is much better to speak about the probability of failure of a certain protection system. In Table G.1.5.11 some possible modes of failure have been listed. All possible causes of failure have to be analyzed and consequences determined. The so-called fault tree is a good tool for this purpose.

Table G.1.5.11 POSSIBLE CAUSES OF FAILURE

Possible failures to be analyzed
Bank instability
Toe scour
Transition between parts or systems
over topping
excessive movement of armour under current attack
excessive movement of armour under wave attack
settlement
loss of sub layer material through armour
loss of subsoil through geotextile filter
loss of grouting or binder materials
deterioration of geotextile filters
failure of cables
failure of pins or other connections
abrasion
corrosion of wire
chemical action
bed lowering by dredging or maintenance
plant growth
cattle
vandalism

An overall fault tree for a bank protection is presented in Figure G.1.5.10. In the following the design will be restricted to the following failure modes, which will be analyzed thereafter:

- geotechnical failure
  - micro instability
  - macro instability
- failure of the slope protection
  - instability of top layers
  - instability of filter layers

The risk analysis will be based on the fault tree presented in Figure G.1.5.10. The specific sites of Bhairab Bazar and Munshiganj has been considered when this fault tree was prepared.

The fault tree is an essential part of the probabilistic design Approach which, as a rule, can only be applied quantitatively at the design stage. The said fault tree is a scheme in which events and their consequences, or errors and their causes, which contribute to the failure, are arranged clearly.

G.1.5.4.3 Acceptable probability of failure

The acceptable probability of failure of an overall bank protection for Bhairab Bazar and Munshiganj is discussed in the following.

In Figure G.1.5.11 a graph frequently used for determination of acceptable risk levels for various structures and activities is shown [1]. Considering the type of protection, magnitude of loads and the commercial areas which are in danger, for both Bhairab Bazar and Munshiganj an acceptable failure probability of 0.5E-03 has been selected.

Starting with this value of a failure probability of 0.5E-03 for a bank protection the failure probabilities of the different components of the fault tree have to be determined. Probabilistic calculations have been performed with Consultants' probabilistic software package HASPROB.

G.1.5.5 **Probabilistic calculations**

G.1.5.5.1 Bhairab Bazar Town and Bhairab Bazar Railway Bridge

(a) Current attack

For the determination of the characteristic diameter of a stone or concrete cement block on a slope, the formula which has been presented in the previous section has been applied for the probabilistic calculations.

In the previous Section it was concluded that according to the prevailing criteria the probability of failure of the element of a bank protection should not be more than indicated for the respective failure modes in Figure G.1.5.10. The reliability function which has been applied can be described by:

$$Z = \Delta_m D_{50}\, 0.847 - \phi K_t \frac{0.035}{\Psi_{cr}} \frac{K_h}{K_s} \frac{\overline{u}^2}{2g}$$

The parameters have been defined as presented earlier.

The average current velocity in a flow profile is derived with the Chezy formula:

$$\overline{u} = C\sqrt{(h\,I)}$$

where:

I = *slope* (-)
C = *Chezy value* $(m^{1/2}/s)$

To determine the waterdepth use is made of the stage relationship as presented in the Hydrological Study, Annex A. For the discharge use is made of a Gumbel extreme value distribution[2] (A=12,971, B=1,814) for the discharges which occur during floods at the Bhairab Bazar area. The values for the extreme value distribution have been obtained from the Hydrological Study, Annex A.

The parameters, except those for the discharges, have a normal distribution, characterised by an average and a standard deviation. From these parameters, other parameters can be derived via simple relationships.

For several combinations of slope, average and standard deviation of D_{50} calculations have been performed (see Appendix G/4). In the following Table the final results are summarized.

---

[2] Gumbel Extreme Value Distribution:

$$P(x < X) = \exp^{\exp\left(-\frac{X-A}{B}\right)}$$

Table G.1.5.12 RESULTS OF PROBABILISTIC CALCULATIONS FOR CURRENT ATTACK AT BHAIRAB BAZAR

Section	Slope	$\mu(D_{50})$ (m)	$\sigma(D_{50})$ (m)	Probability of failure (1/year)	Acceptable probability of failure (1/year)
Falling apron	1:2	0.15	0.015	3.69*10-3	3.12*10-2
Lower part	1:3.5	0.15	0.015	2.22*10-4	3.12*10-2
Upper part	1:3.5	0.15	0.015	2.26*10-4	6.25*10-3

The calculated probabilities show that the acceptable failure probabilities are not exceeded.

(b) Wave attack

In Section G.1.3.2.5 information on wind velocities and wave heights is presented for several return periods. It is assumed that the probability density function for the wave heights can be described by a Gumbel extreme value distribution (A=0.546, B=0.094). For determination of the characteristic diameter of a stone on a slope, the formula for wave attack which has been presented in section 5.3.4 has been applied for the probabilistic calculations. Slightly rewritten this formula is as follows:

$$Z = D - \frac{H_s}{6.0}$$

The main results of the probabilistic calculations (see Appendix G/4) are presented in the following Table.

Table G.1.5.13 RESULTS PROBABILISTIC CALCULATIONS WAVES (OPEN STONE ASPHALT)

Slope	$\mu(D_{50})$ (m)	$\sigma(D_{50})$ (m)	Probability of failure (1/year)	[3]Acceptable probability of failure (1/year)
1:3.5	0.15	0.015	1.05*10-1	1.5*10-1

As shown in the Table the probability of failure does not exceed the permissable one. Therefore Consultants adopt dimensions as presented in Table G.1.5.13.

In view of above results Consultants selected for the thickness of the open stone asphalt a thickness of 0.15 m.

(c) Scour depths

The design of the bank protection shall be based on a combination of various forms of scour. Use has been made of the formula presented in Section G.1.4.2.1.

For determining the probability of failure of a falling apron for a certain scour depth the reliability function which has been applied is based on the aforementioned formula.

For the discharge use is made of a Gumbel extreme value distribution. The other parameters involved follow a Gauss distribution.

---

3 The value presented is different from the one presented in the fault tree. A commonly used criterion for wave attack at open stone asphalt is 1.5*10-1

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The following scour depth along the bank protection and corresponding probability of failure has been determined. The results are listed below.

Table G.1.5.14 RESULTS PROBABILISTIC CALCULATIONS EXPECTED SCOUR DEPTHS

Scour depth referred to initial bed level (m)	Probability of failure (-)	Acceptable probability of failure (-)
11.50	8.12 10-2	1.20 10-1

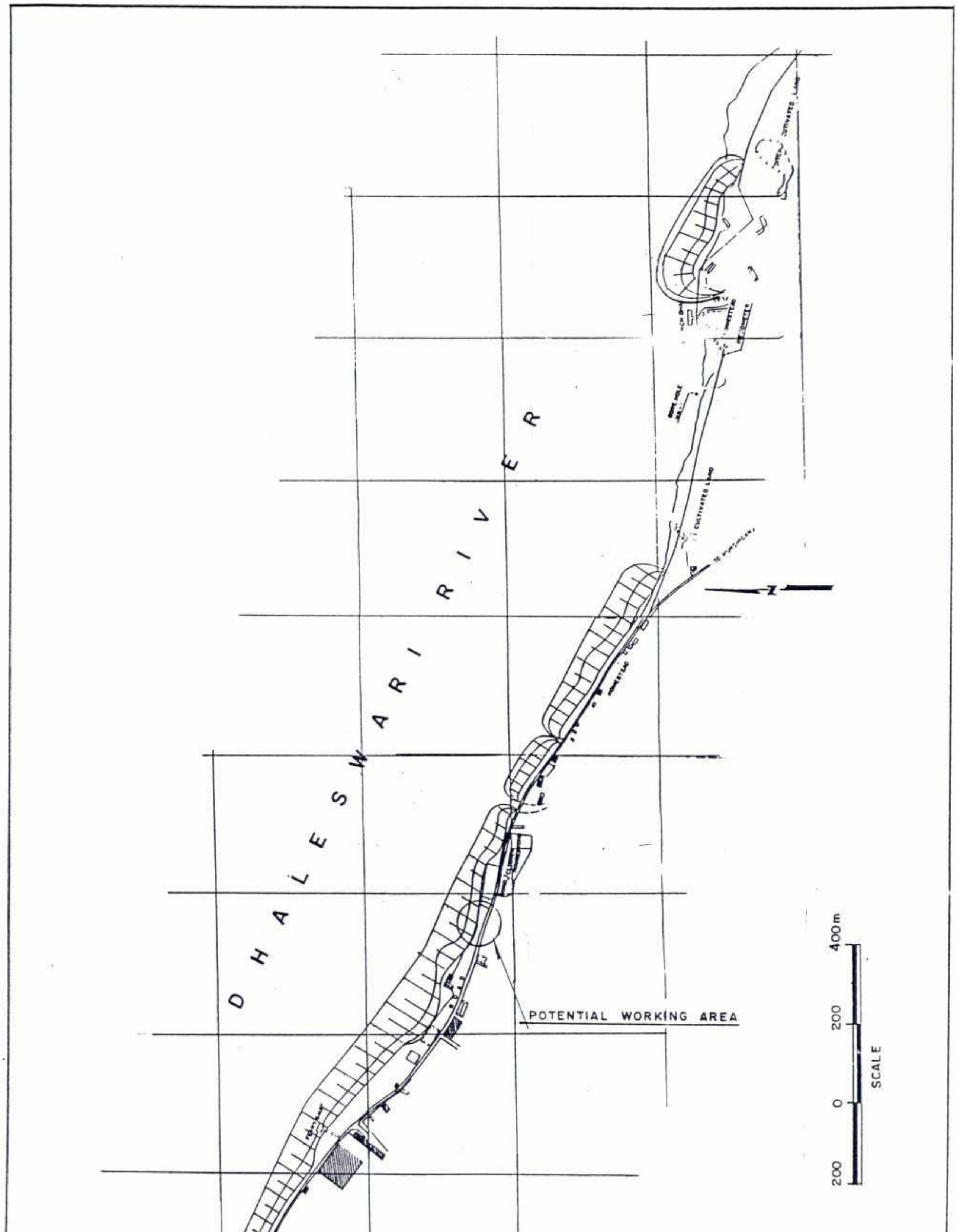
As shown in the Table the probabilities of failure are lower than the acceptable ones. In Appendix G/4 more results of the probabilistic calculations are presented.

The dimensions for the falling apron, as presented in previous Sections, are the same for the entire bank protection works along Bhairab Bazar.

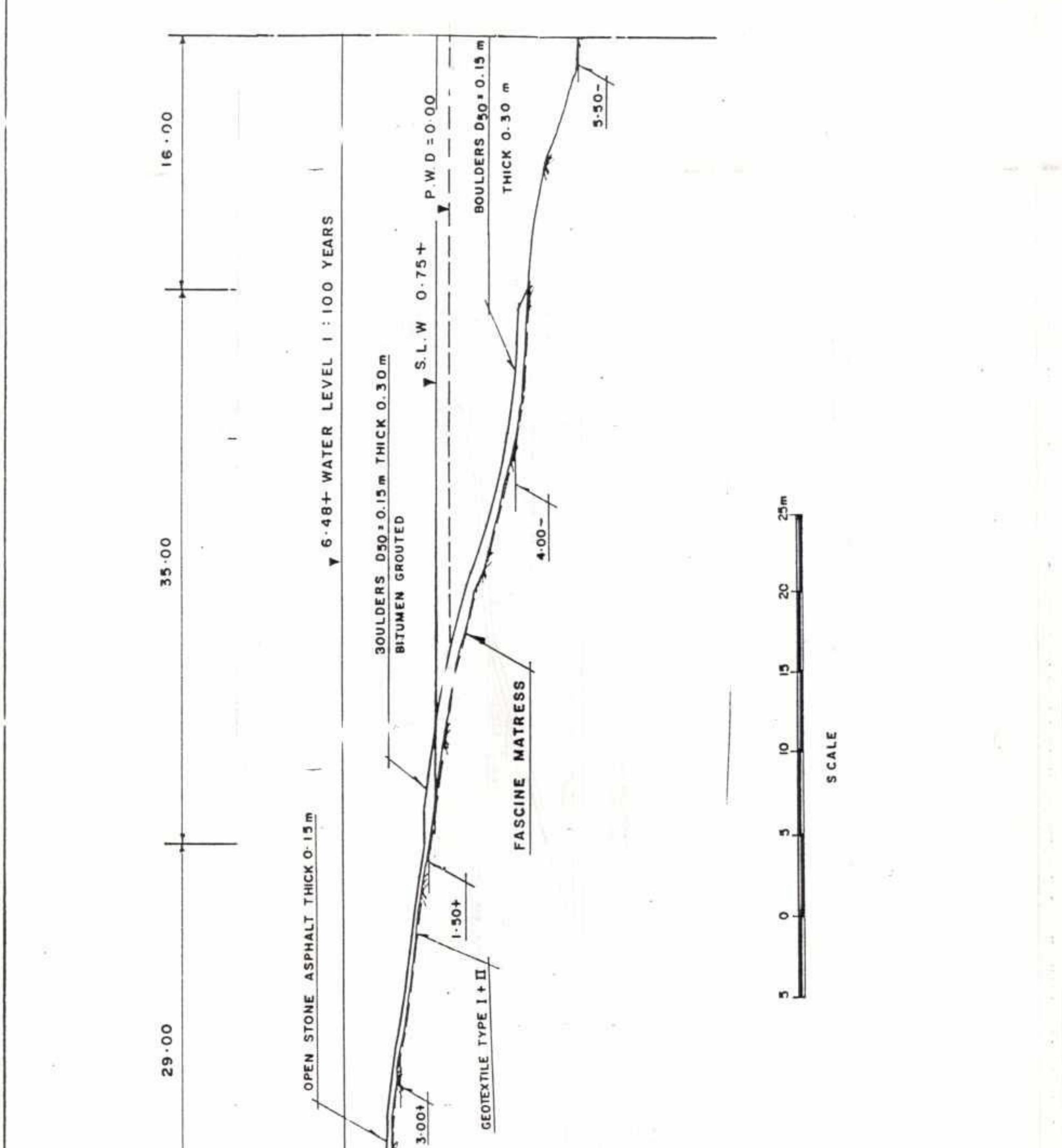
For the design level of the falling apron a scour depth of 11.50 m referenced to the initial bed level has been selected.

G.1.5.5.2 Munshiganj

In view of the similarity of the stochastic parameters for the loads at Munshiganj compared to Bhairab Bazar, Consultants are of the opinion that the protection as proposed for the former location will have lower probabilities of failure than the acceptable ones.

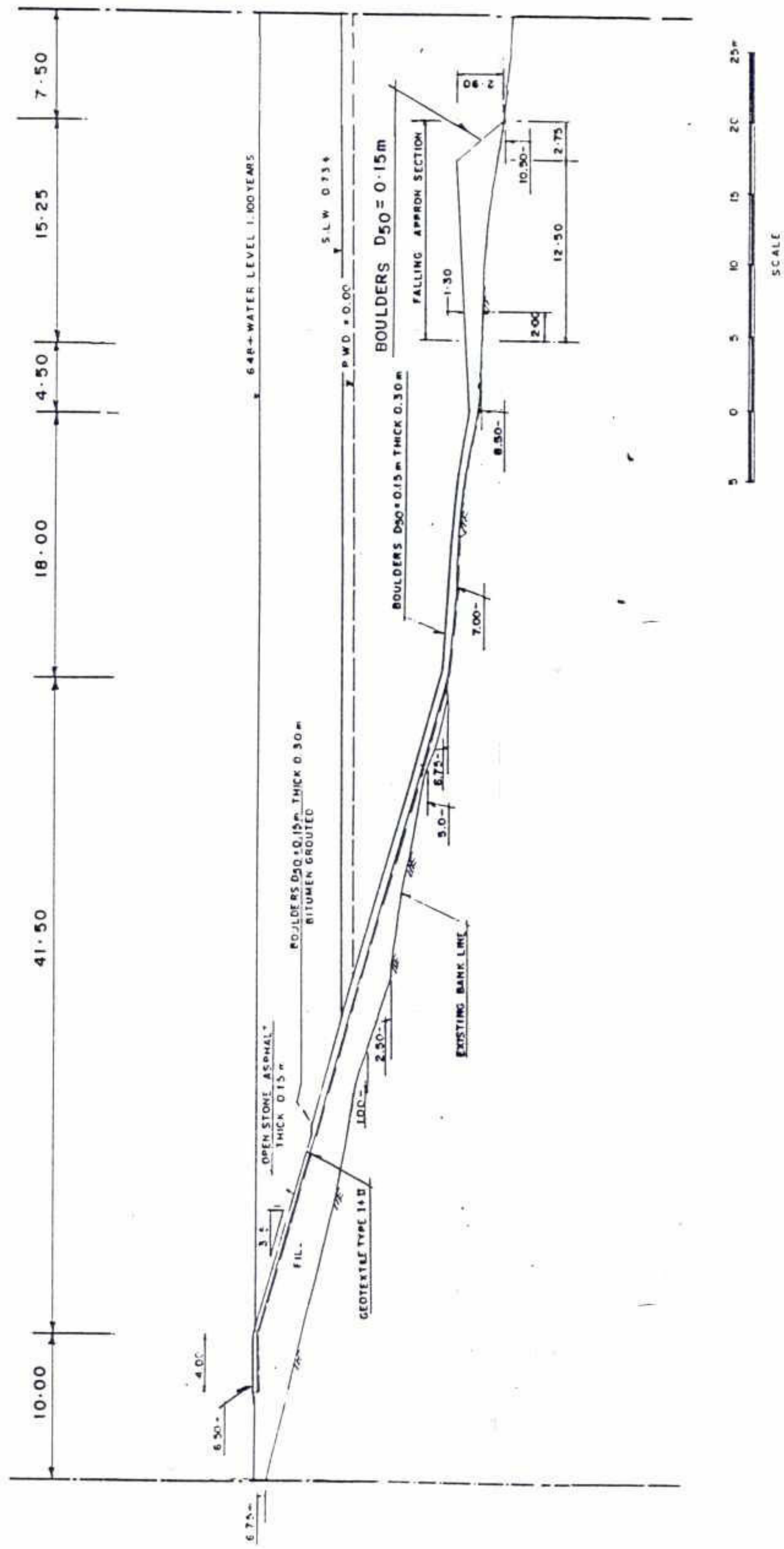


LAYOUT PLAN MUNSHIGANJ PROTECTION | FIGURE G.1. 5.1



TYPICAL CROSS SECTION MUNSHIGANJ

FIGURE G.I. 5.2



TYPICAL CROSS SECTION MUNSHIGANJ

FIGURE G.I.5.3

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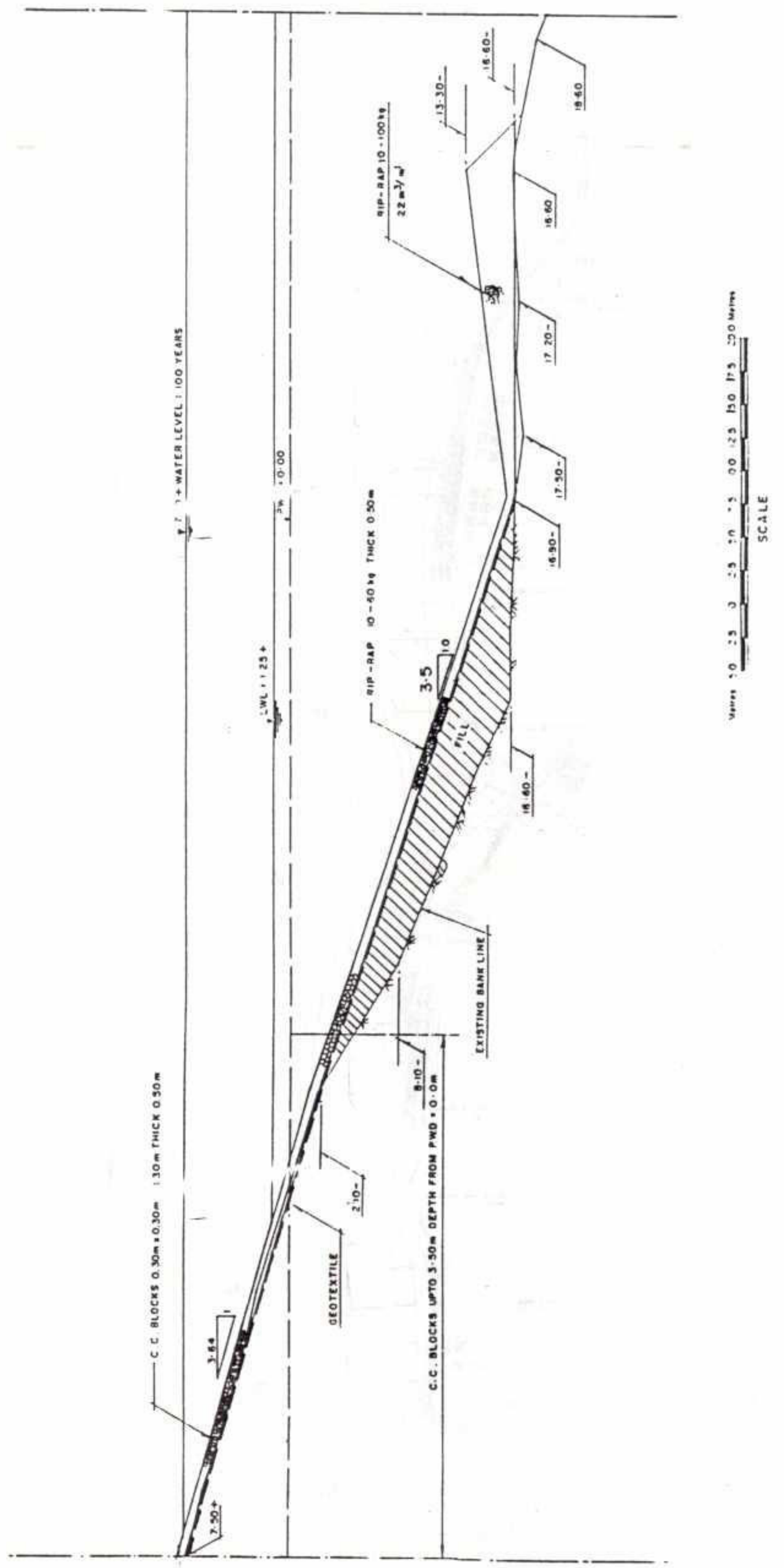

TYPICAL CROSS SECTION BHAIRAB BAZAR PROTECTION ON EXISTING SLOPE	FIGURE G.I.5.4

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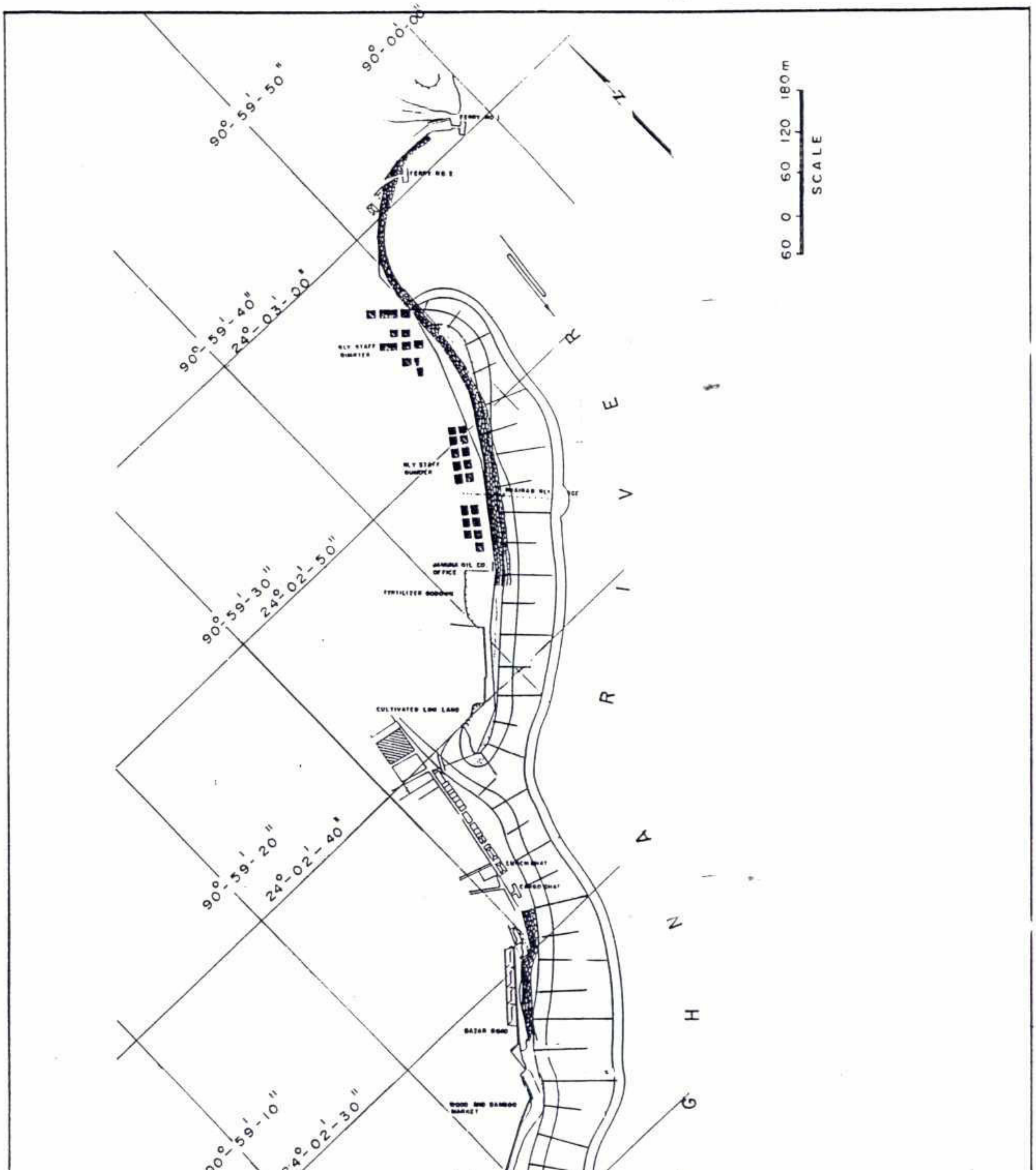
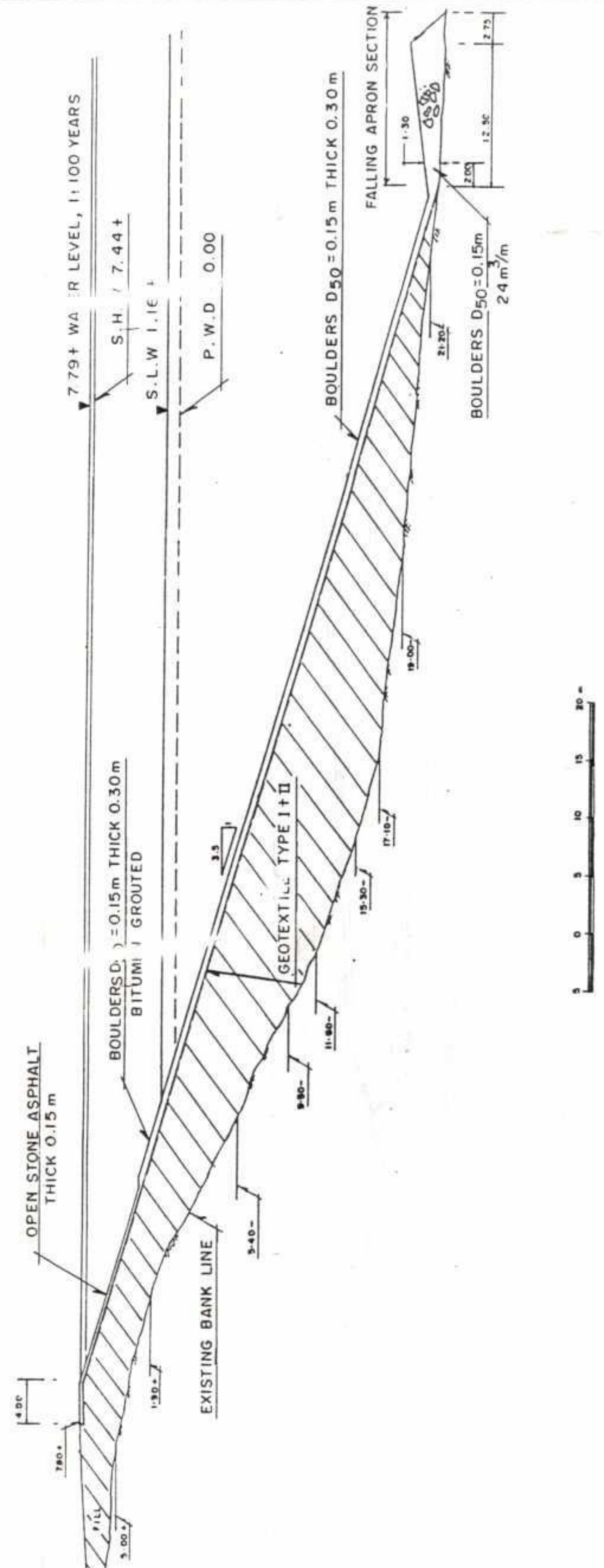


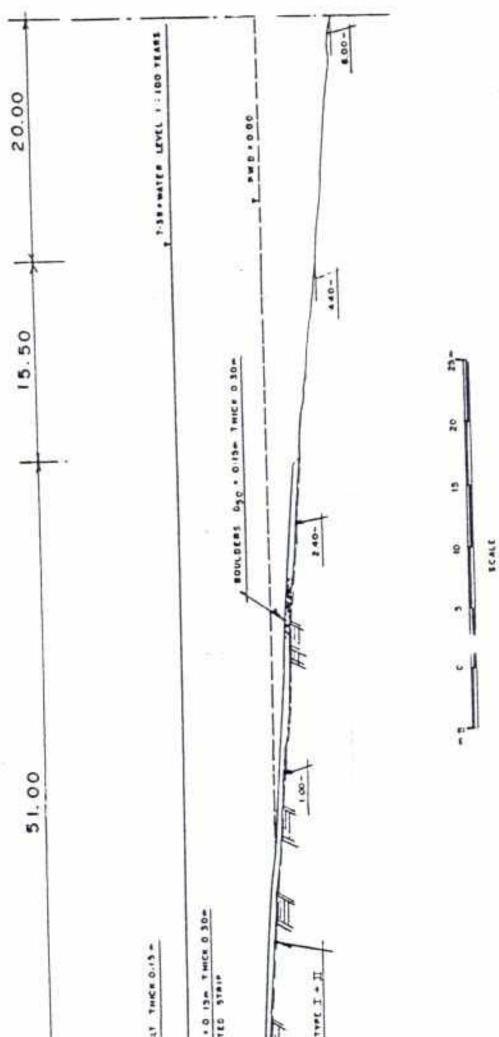
LAYOUT PLAN BHAIRAB BAZAR
ADVANCED PROTECTION

FIGURE G.1.5.5

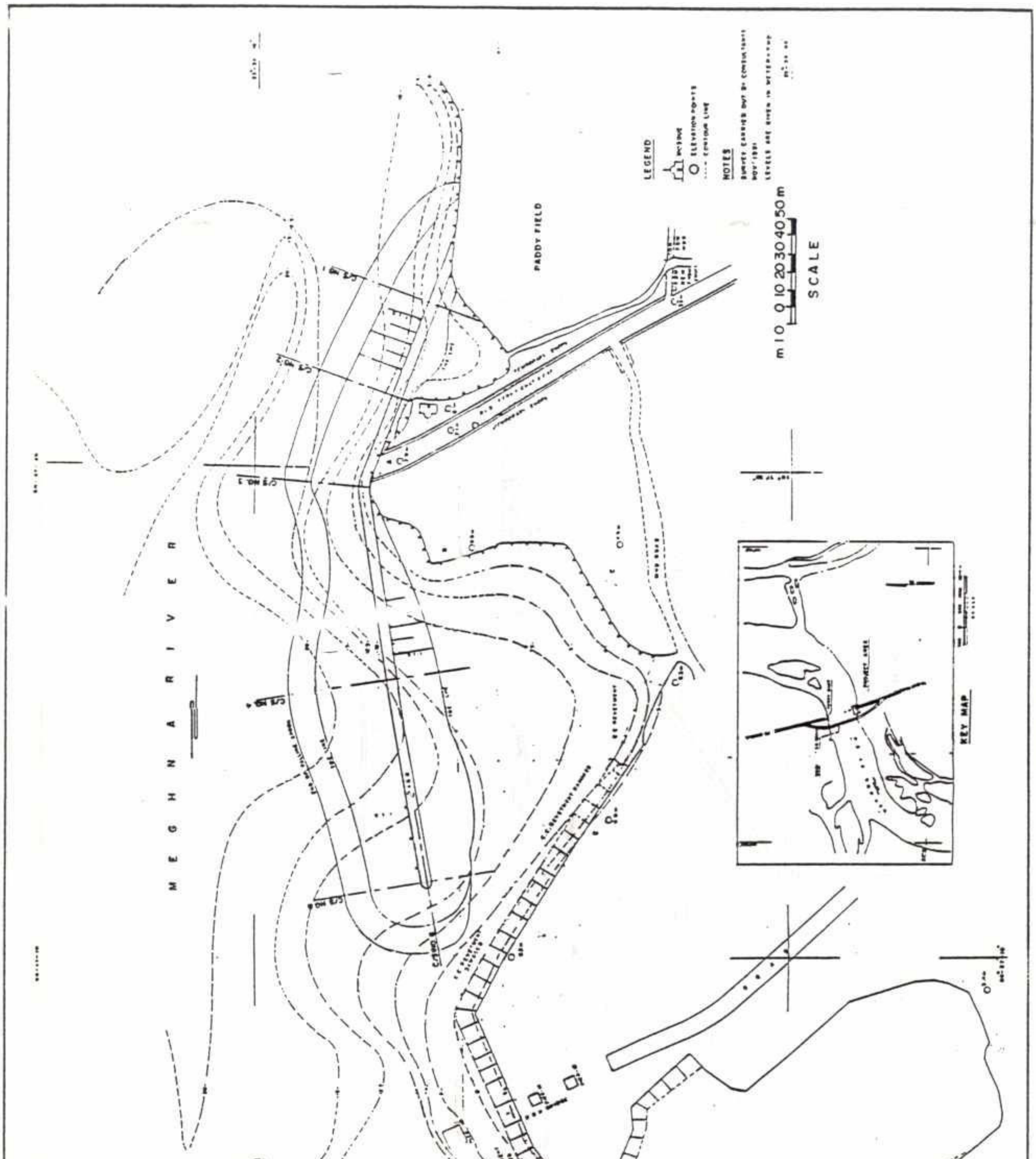


TYPICAL CROSS SECTION BHAIRAB BAZAR (ADVANCED PROTECTION)

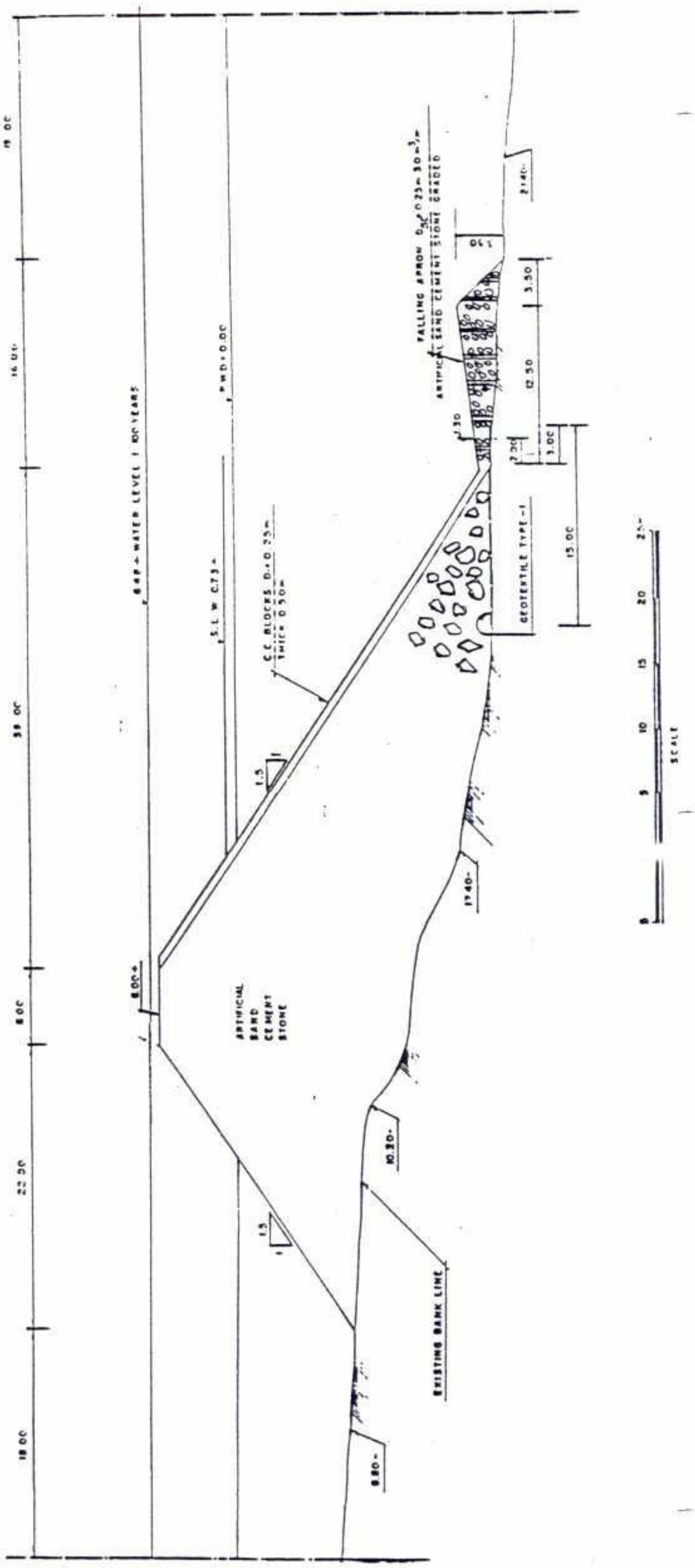
FIGURE G.I. 5. 6



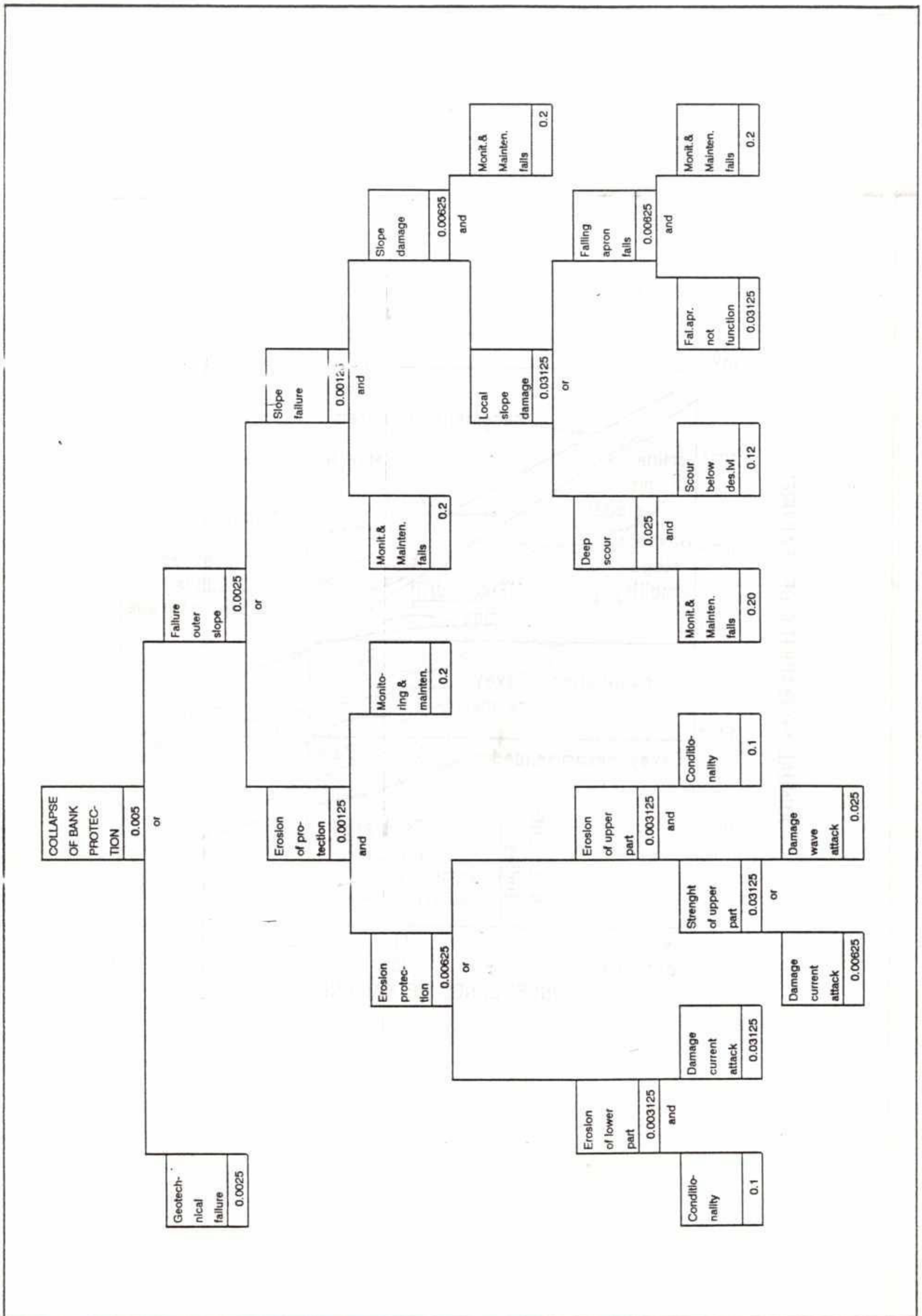
TYPICAL CROSS SECTION MANIKNAGAR | FIGURE NO. G. I.5.7



LAYOUT ROADS AND HIGHWAYS BRIDGE	FIGURE NO. G. I. 5.8



TYPICAL CROSS SECTION ROADS AND HIGHWAYS BRIDGE	FIGURE NO. G.1.5.9



FAULT TREE FOR BHAIRAB BAZAR

FIGURE G.I.5.10

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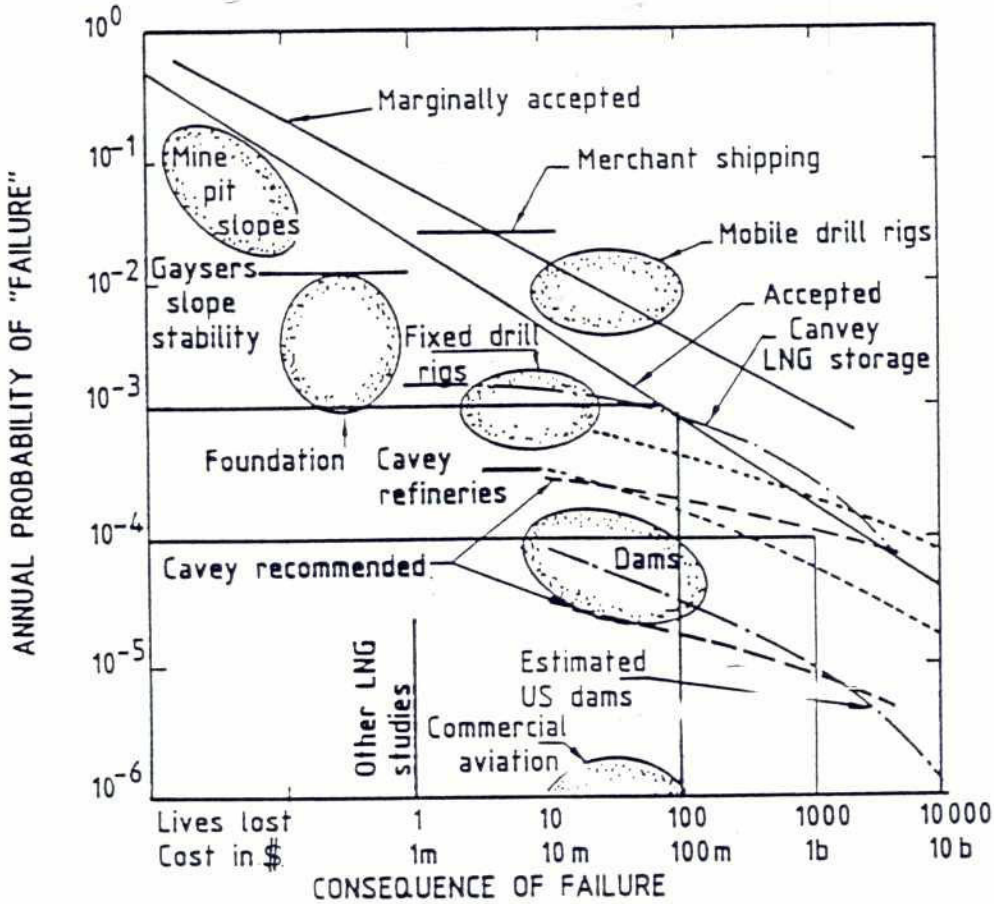

CRITERIA FOR FAILURE PROBABILITIES | FIGURE G.1.5.11

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G.1.6 CONSTRUCTION METHODS

G.1.6.1 General

The sites along the Upper Meghna for which final designs have been prepared are Bhairab Bazar and Munshiganj. Two vital elements in the con .ruction of new embankments or revetments at these sites are: dredging and protection of slopes under water. The works have more in common: both should be carried out in a relatively short ti... ..ainly around the dry season. Preparations should be made in the time preceding the dry season.

The inter-relation between the various construction activities is very critical. For instance the re-dredging of a slope at Bhairab Bazar should be followed immediately by the installation of slope protection mattresses. It is obvious that such a project can only successfully be completed by contractors who not only have experience with all the elements of the work, but also have the capability to plan and coordinate all those elements.

There are no contractors in Bangladesh who have all the resources and experience to complete the works on their own, even when they would join forces. It is therefore inevitable that the works will be executed under the responsibility and control of a foreign contractor with ample working experience in similar works and with adequate equipment at hand or at his disposal.

Yet for the designs of the new embankments and slope protection works every effort has been made to use as many local resources as possible. Also Bangladeshi contractors will have ample opportunity to participate in the construction works, mainly in the capacity of suppliers and sub-contractors.

G.1.6.2 Dredging and reclamation

(a) Bhairab Bazar

To store his materials ... to prepare slope protection mattresses, the Contractor requires an adequate working area. Inspection of the site has revealed that there are no suitable areas available at the right bank. On the other hand there appears to be space at the left bank, just upstream of an area not so long ago reclaimed for Petro Bangla. The Contractor's work area would have to be reclaimed with hydraulic fill from the river. Filling can be done using a cutter suction dredger.

The soil for the advanced protection at Bhairab is foreseen as hydraulic fill. This fill could be placed by a cutter suction dredger, or by a small trailing suction hopper dredger. The under water slope of the hydraulic fill might not be steeper than 1:7, which is too gentle for the slope protection (too large an area to be protected) and would reduce the cross sectional area of the Upper Meghna too much. Therefore the underwater slopes will have to be re-dredged, as short as possible prior to the placing of the fascine mattresses. Only a well controlled cutter suction dredger is suitable for shaping the under water slopes. As it is very costly to mobilise two different dredgers for the relatively small quantity of dredging work, it is likely that the contractor will also use a cutter suction dredger for the other dredging activities.

(b) Munshiganj

For Munshiganj there are few options other than to obtain the required quantity of earth fill by dredging. In view of the relatively high level of river bed adjacent to the bank, the fill may have to be placed initially in one or more soil storage areas, from where it can be transported to its final destination using dry earth moving equipment. In this case the need to re-dredge too gentle slopes, which would result from under water disposal of dredged soil, can be avoided, thus also avoiding the need to have a dredger on site for a very long time.

G.1.6.3 Slope protection mattresses

Generally, slope protection mattresses consist of a geotextile fabric with a cover of boulders. For bringing the geotextile in place it will be necessary to prepare mattresses on a launching ramp. Bamboo fascines have to be fixed to the geotextile to arrive at sufficient buoyancy (necessary during transport) and flexibility, without folding, etc. during the sinking of the mattress. Experience with the Feni River Closure Dam has learned that the sinking can successfully be carried out making use of almost exclusively local resources, including labour, provided the management thereof is very strong. In Figure G.1.6.1 a detail of a fascine mattress is shown.

G.1.6.4 Falling apron

Boulders for the falling apron can be applied using the same equipment and labour resources which will be required for the dumping of boulders on the slope protection mattresses.

G.1.6.5 Open stone asphalt

Open stone asphalt is a material which has probably not been used before in Bangladesh. Yet it can be made using fairly standard asphalt production plant, which is probably available in Bangladesh, and almost exclusively local materials. The skills of making the upper part of revetments using this material can probably be transferred to Bangladeshi contractors in the course of the project. In Figure G.1.6.2 a detail of the open stone asphalt is shown.

G.1.6.6 Containment bunds

When backfilling under water conditions must be created to ensure that optimum density of deposited material is achieved. This may require the use of sand bags (jute) and/or backfilling methods in a sheltered condition. Deformations will be negligible.

Construction of an "advanced" protection in front of Bhairab Bazar making use of hydraulically placed fill under water, and using a "safe" slope gradient of 1:3.5 may lead to post-construction settlements in case of liquefaction caused by an earthquake. Attention should be given to this phenomenon in the risk analysis (see Annex C). Should liquefaction occur then this may theoretically lead to a mass flow. However case histories of mass flows, particularly in the Netherlands (where loosely packed fine sands can be found in the southern provinces), indicate that such mass flows only occur when steep slopes (say 1:1.5) over a substantial height are present along the length of an earth structure, like a dike or dam. The proposed slope gradient for the Meghna project (1:3.5) would appear to be sufficiently conservative in this respect.

If, for the construction under water, containment bunds are considered (see Figure G.1.6.3), highly sophisticated equipment will be required. Two types of floating equipment then come in mind: a stone dumper with highly controlled sideways dumping or a work-ship equipped with a fall pipe; the end of the fall pipe may need to be provided with a remotely controlled vehicle. Yet use of such equipment may be the only acceptable possibility to achieve the required accuracy and to minimize the use of granular material. Use of less accurate equipment can easily lead to a substantially larger quantity of coarse material being required.

For placing of the bulk of the hydraulic fill use can be made of a cutter suction dredger or a self trailing suction hopper dredger. In the latter case a small cutter suction dredger will still be required to place hydraulic fill in the upper part of the new embankment, unless the hopper dredger would be provided with a facility to pump fill material ashore.

fs

G.1.6.7 **Construction programmes**

The construction program for the advanced protection at Bhairab Bazar and Munshiganj is presented in Figure G.1.6.4. It is assumed that construction of protection works at these sites will be tendered as one package. This is reflected in the construction program.

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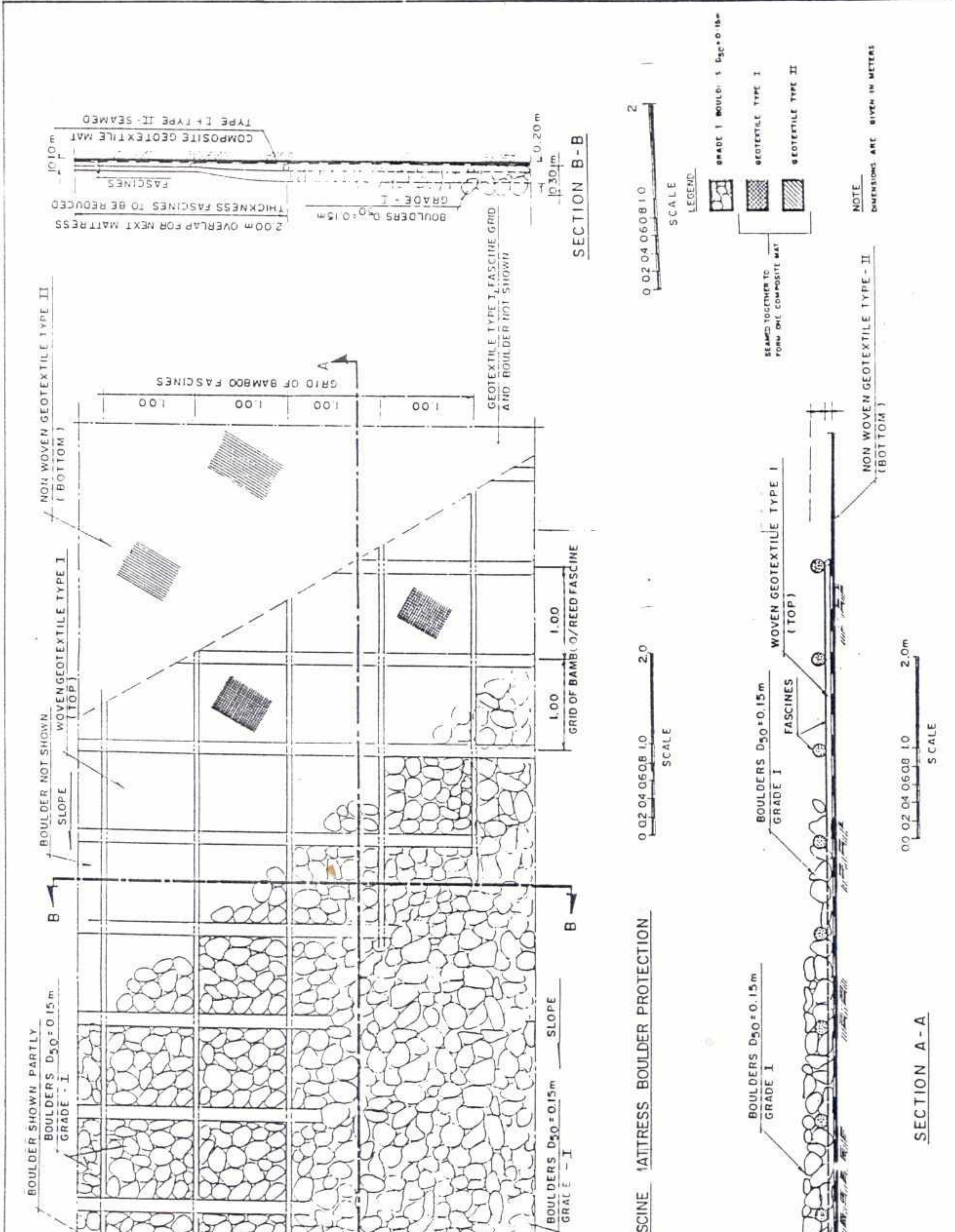

TYPICAL FASCINE MATTRESS

FIGURE NO. G.I. 6.1

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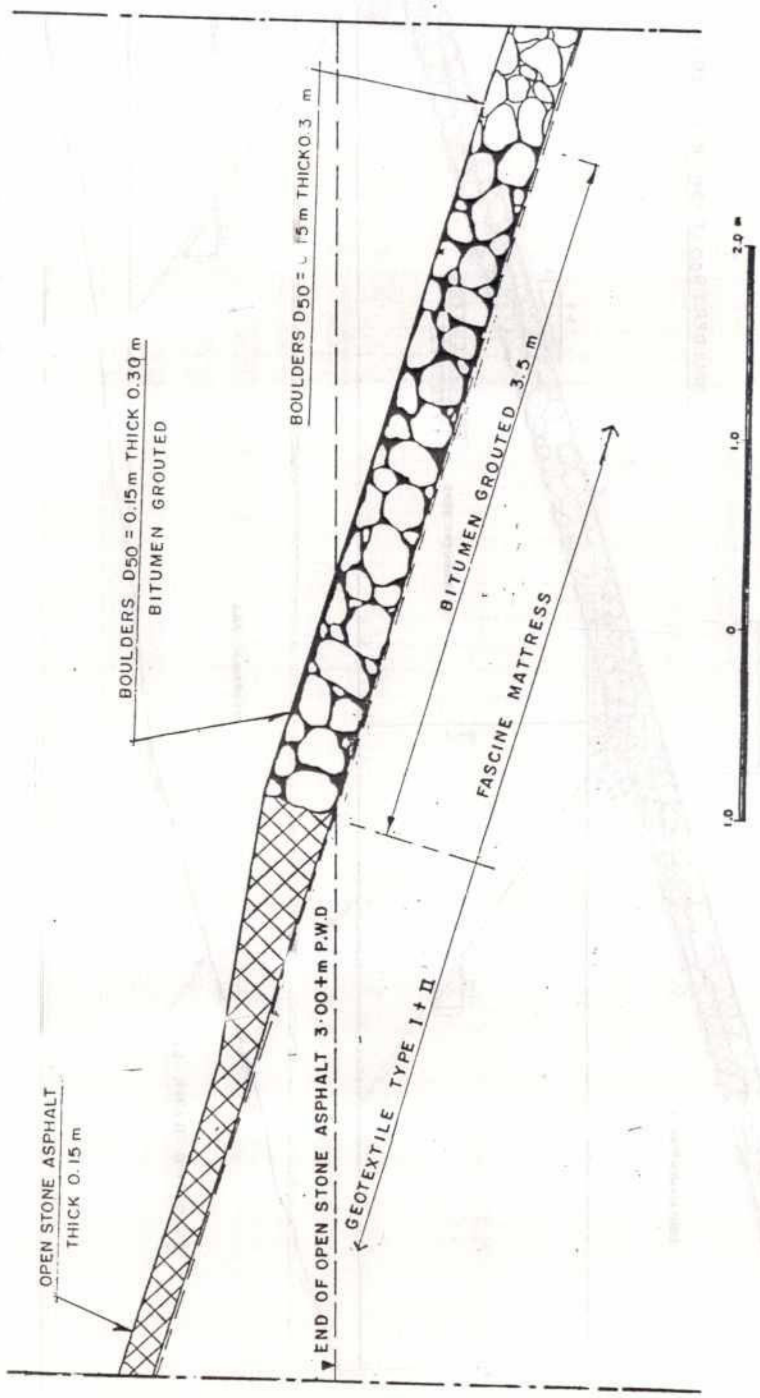

DETAIL OPEN STONE ASPHALT	FIGURE G.I.6.2

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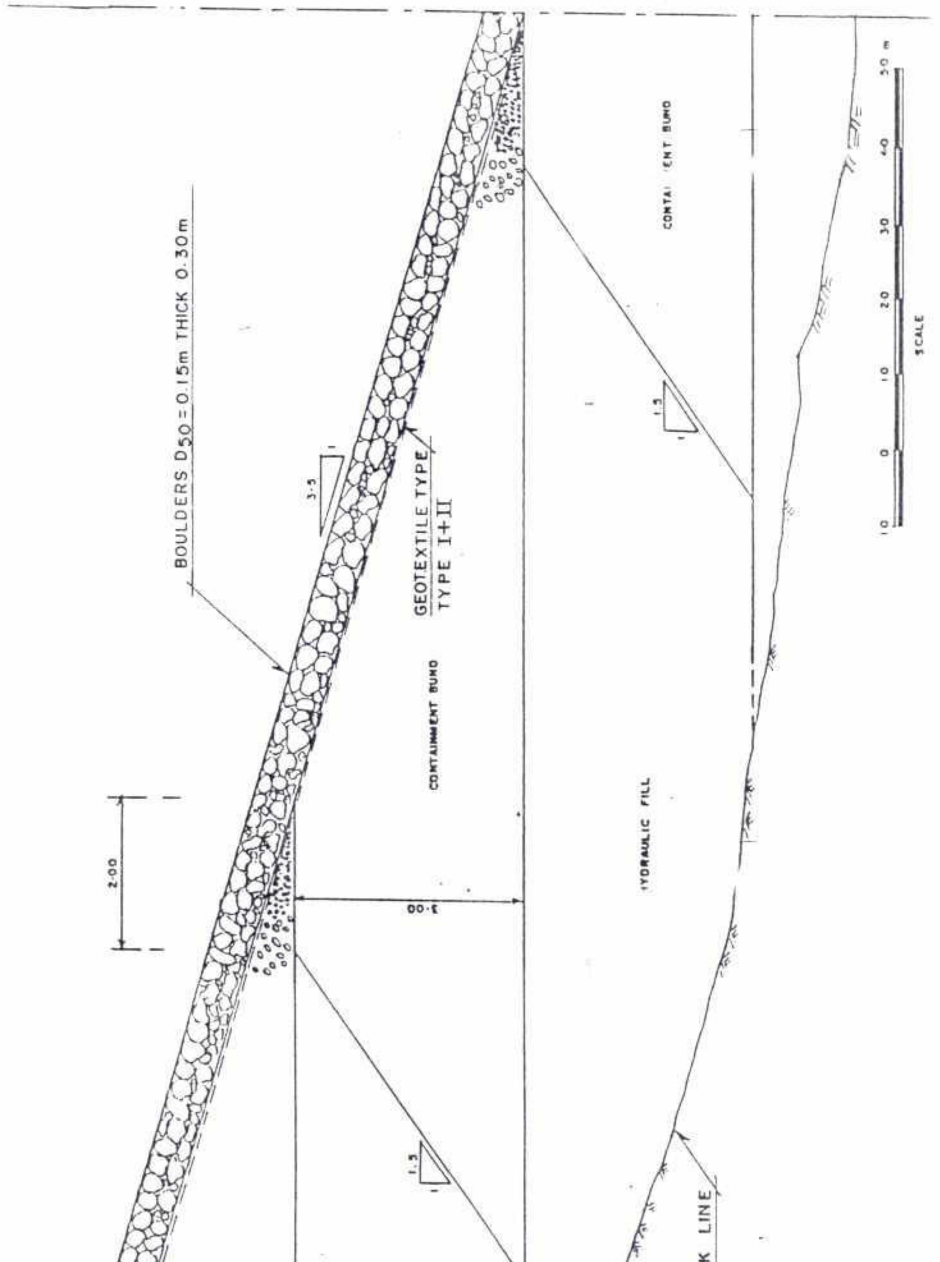
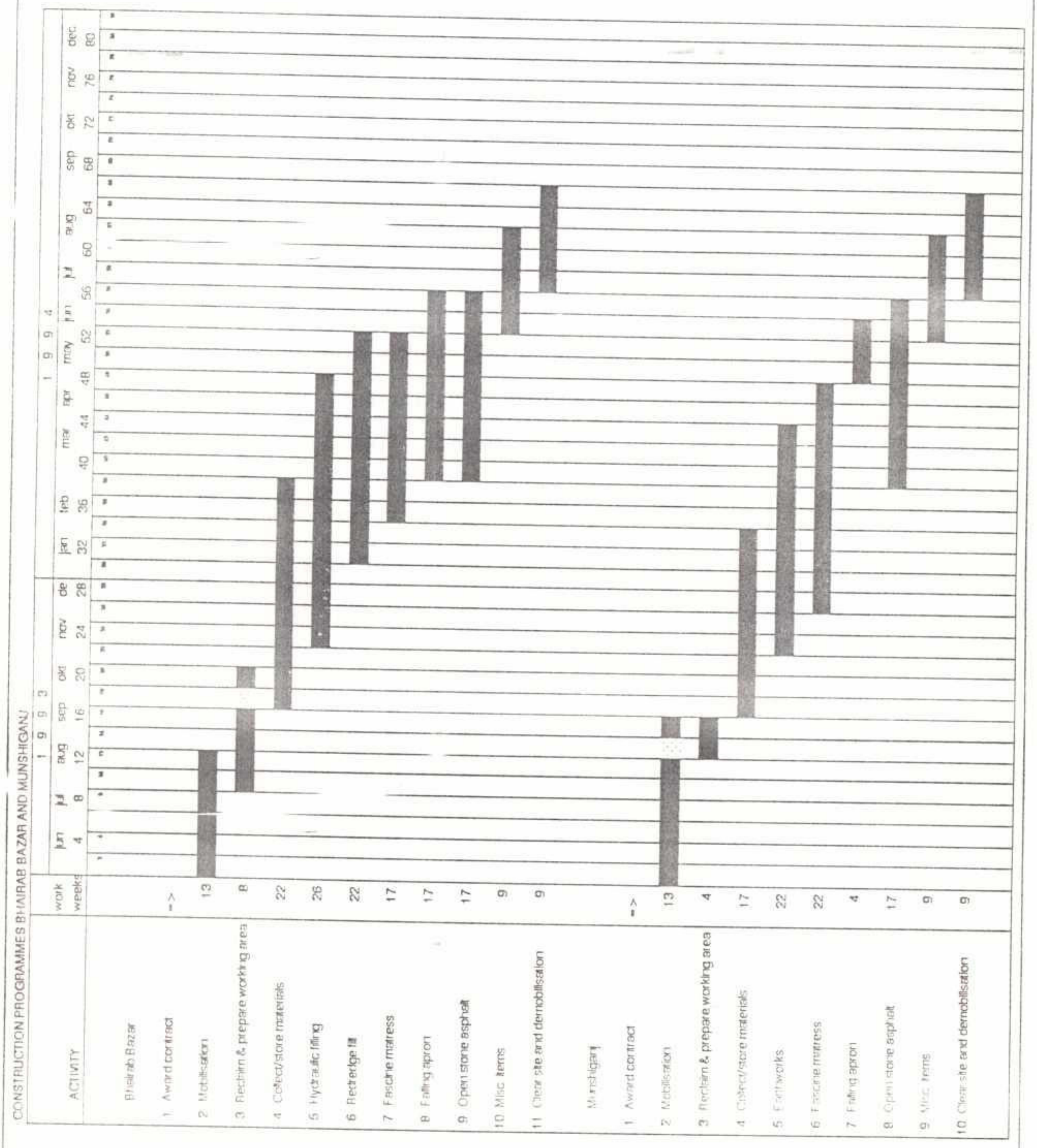


CONTAINMENT BUNDS	FIGURE G.1.6.3



CONSTRUCTION PROGRAM BHAIRAB BAZAR AND MUNSHIGANJ

FIGURE G.1.6.4

G.1.7 COST ESTIMATE

G.1.7.1 Cost

G.1.7.1.1 Bhairab Bazar Town and Bhairab Bazar Railway Bridge

Cost comparisons have been made for the alternative solutions for Bhairab Bazar. In the following Table these comparisons have been given based on engineers cost estimates.

Table G.1.7.1 COST ESTIMATES FOR VARIOUS ALTERNATIVE SHORT TERM MEASURES FOR BHAIRAB BAZAR

No.	Description	Cost in US$
1	Protection of existing slope.	13.900.000
2	Advanced protection.	15.267.440

For the advanced protection the breakdown of the cost estimate is presented.

Table G.1.7.2 COST ESTIMATE ADVANCED PROTECTION BHAIRAB BAZAR

Item	Quantity	Unit	Unit cost (US$)	Cost (US$)
1 Dredging	1,550.000	m3	3.76	5.824,207
2 Working area/materials	1	-	138,358	138,358
3 Earthworks above SLW	1	-	331,128	331,128
4 Clear site and reinstate	1	-	23,652	23,652
5 Open stone asphalt	36,000	m2	22.77	819,713
6 Fascine Mattress	110,000	m2	15.89	1.747,510
7 Boulders in falling apron	41,000	m3	34.34	1,411,634
8 Grouting of boulders	1,700	m	62.58	106,390
9 Contractors cost and supervision	1	-	641,000	641,000
10 MOB and DEMOB	1	-	[illegible]	60,000
				=======
				11.103.593
11 Physical contingencies	10	%		1,110,359
12 Contractors margins and fees	20	%		2,220,719
13 Engineering and supervision	7.5	%		832,769
				======
TOTAL				15.267,440

The cost per linear meter for the protection works at Bhairab Bazar is 8.981 US$/m.

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G.1.7.1.2 Munshiganj

For Munshiganj the engineers cost estimates are presented in the following Table.

Table G.1.7.3 COST ESTIMATES FOR PROTECTION WORKS MUNSHIGANJ

Item	Quantity	Unit	Unit cost (US$)	Cost (US$)
1 Dredging	530.000	m3	3.36	1.779.619
2 Working area/materials	1	-	148.436	148.436
3 Temporary access diversions and culverts	1	-	123.812	123.812
4 Earthworks	1	-	509.432	509.432
5 Open stone asphalt	31,090	m2	22.77	707,913
6 Fascine Mattress	83,100	m2	15.89	1,320,165
7 Boulders in falling apron	5,760	m3	34.38	198,317
8 Grouting of boulders	1,875	m	62.58	117,342
9 Clear site and reinstate	1	-	46,312	46,312
10 Contractors cost and supervision	1	-	607,650	607,650
11 MOB and DEMOB	1	-	80,000	80,000
				=======
				5,321,654
12 Physical contingencies	10	%		563,900
13 Contractors margins and f[illegible]	20	%		1,127,800
14 Engineering and supervision	7.5	%		422,925
				======
TOTAL				7,753,623

The cost per linear meter for the protection works at Munshiganj is 4.135 US$/m.

G.1.7.1.3 Maniknagar

For the alternatives cost estimates have been made for the protection works of Maniknagar. They are summarised in the following Table.

Table G.1.7.4 PRELIMINARY COST ESTIMATES PROTECTION WORKS MANIKNAGAR

Item	Alternative	Cost in US$
i	Overall protection	12,964,717
ii	Series of groynes (earth filling)	11,838,105
iii	Series of sand sausages	20,551,285
iv	Deviation of flow with groyne	9,142,974

These costs have been used in Annex F, Economics of Protection Works in order to calculate the EIRR. As discussed in Annex F the alternative of 'doing nothing' in the near future has been selected.

G.1.7.1.4 Roads and Highways Bridge

Cost estimates have been made for the various alternatives of protection works at Roads and Highways Bridge. They are summarised in the following Table.

Table G.1.7.5 COST ESTIMATES PROTECTION WORKS R&H BRIDGE

Item	Alternative	Phase	Construction in year	Cost in US$	Total cost in US$
i	protection of ferry ghat and vortex area	no phasing	1992	5.207.667	5.207.667
ii	Protection of ferry ghat and vortex area and groyne of 200m upstream of bridge	protection of ferry ghat and vortex area	1992	5.207.667	16.014.553
		groyne of 200 m	2003	10.806.886	
iii	Spurdike which guides flow lines (toplayer of CC-blocks)	no phasing	1992	5.570.846	5.570.846
iv	Spurdike which guides flow lines (toplayer of sand cement stone blocks)	no phasing	1992	5.155.115	5.155.115

In the Table also the investment scheme for the second alternative has been presented (see also Annex F, Economics Protection Works). The advantage of phasing the investments is a higher value of the EIRR, see Annex F. The alternative selected has been underlined.

G.1.7.2 Maintenance

The bank protection works will be subjected to variable loads, the most severe of which are likely to occur during high river stages. With probabilistic methods, the "skin" of the river training works has been designed so strong that the risk of failure falls within acceptable limits (established in the risk analysis exercise). Yet local failures of, notably, the "skin" may occur. Also the behaviour of the falling apron may be different from that was assumed in design calculations despite all the precautions taken in the design methods and procedures. The location where such failure may materialise or a different "falling behaviour" becomes apparent is unpredictable. Therefore it is essential that regular inspection takes place, so that any damage or irregularities will be noticed within reasonable time. Corrective action can then be taken, so that the damage will be contained. Progressive erosion of the bank protection, following local failure of the protective "skin" or unexpected "falling behaviour" would thus be prevented. The material cost for maintenace amount to 10% of the investment cost and for this purpose it is required to have in stock construction materials for a period of 3 years. (see also Annex F, Economics of Protection Works)

In the detailed design stage due consideration has been given to possible methods for the detection of local damage, see also Appendix G/3. Complicating factors in this respect will be the high turbidity of the Meghna River water and the high current velocities, particularly during the flood season. Monitoring can be done with suitable survey equipment and river craft. This would not only be beneficial for the safety of the bank protection, but also for achieving an improved insight in the behaviour of the Meghna River, particularly in the "high flow" season.

Despite the high current velocities during floods it is considered possible to obtain information on localised damage by using high accuracy (double frequency) echo sounders, coupled to a sophisticated positioning system. Survey data should collected in digital form and should be transferred to a (correct) data bank. Data in the data bank should then be used to make regular plots of the river training works. As the data are digitalized it should be relatively easy to obtain also "differential" plots, so that possible damage, but also erosion and accretion near the river training works could be "tracked".

Apart from damage emanating from the environmental loads (for which the bank protection works have been designed), shifting of existing and development of new river channels has to be closely monitored. To this end a regular overview of the Meghna River has to be made, for which satellite imageries will be indispensable. Such overview can best be made at yearly intervals, making use of imageries for low river.

G.1.8 ANALYSIS AND SELECTION OF ALTERNATIVES BHAIRAB BAZAR

G.1.8.1 Application of Multi Criteria Analysis

Two alternatives have been identified as being suitable for short term protection works for Bhairab Bazar.

These alternatives are:

Alternative I Overall protection on existing embankment and protection between piers:

Alternative II Overall protection on advanced embankment and protection between piers.

With the help of a Multi Criteria Analysis (MCA), briefly described in Appendix G/1, the most objective choice will be made between these two alternatives. In the following Section the selection of the criteria is discussed.

G.1.8.2 Selection of Criteria

The selection of criteria which are of diverse nature, such as structural, social and economic aspects, is difficult to make in an objective way. A Multi Criteria Analysis (MCA) method however renders it possible to get more insight into the various relevant aspects. MCA gives a framework to judge primary and secondary criteria and gives the relative importance of the criteria. In Table G.1.8.1 the primary criteria are listed whereas in Table G.1.8.2 also the secondary criteria are listed.

To determine the norm values each code is considered and the criteria are judged by giving a mark in the range from 1 to 3. The results are listed in the following Table.

Table G.1.8.1 DETERMINING NORM VALUE FOR PRIMARY CRITERIA

	Flexi-bility	Dura-bility	Construc-tion	Mainte-nance	Environ ment	Human factors	Σ	Weigh-ting factor
Flexi-bility	0	2	3	3	3	3	14.00	0.23
Dura-bility	2	0	3	2	3	2	12.00	0.20
Construc tion	1	1	0	3	3	3	11.00	0.18
Mainte-nance	1	2	1	0	2	2	8.00	0.13
Environ-ment	1	1	1	2	0	2	7.00	0.12
Human factors	1	2	1	2	2	[illegible]	8.00	0.13
							60.00	0.99

where:

1 = row criterion is more important than column criterion
2 = both criteria are equally important
3 = column criterion is more important than row criterion

G.1.8.3 Evaluation of alternatives

For each of the Alternatives a mark Y will be given indicating the suitability of the Alternative for each criterion. The results are listed in Tables G.1.8.2 and G.1.8.3 for Alternatives I and II respectively.

Table G.1.8.2 CALCULATION OF SCORE FOR ALTERNATIVE I

Primary criteria	Z (%)	Secondary criteria	X (%)	Y	W
Flexibility	24	Settlements	20	2	9.60
		Scour	40	2	19.20
		Geotechnical	40	1	9.60
Durability	20	Erosion	40	3	24.00
		Climate	30	2	12.00
		Chemicals	15	3	9.00
		Biologic	15	2	6.00
Construction	18	Duration	40	2	14.40
		Availability	20	0	0.00
		Quality control	40	1	7.20
Maintenance	13	Monitoring	40	1	5.20
		Duration	20	2	5.20
		Replacement	40	1	5.20
Environment	12	Pollution	40	2	9.60
		Impact	50	1	6.00
		geometry/colour	10	1	1.20
Human Factors	13	Vandalism	10	2	2.60
		Social impact	60	0	0.00
		Mishaps	30	1	3.90
		TOTAL			149.90

where:
X = weight of secondary criteria in %
Y = suitability of alternative in points
- Y=0 satisfies requirements almost not at all to poorly
- Y=1 satisfies requirements poorly to sufficiently
- Y=2 satisfies requirements sufficiently to reasonably
- Y=3 satisfies requirements reasonably to well

Z = weight of primary criteria in %

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Table G.1.8.3 CALCULATION OF SCORE FOR ALTERNATIVE II

Primary criteria	Z (%)	Secondary criteria	X (%)	Y	W
Flexibility	24	Settlements	20	2	9.60
		Scour	40	2	19.20
		Geotechnical	40	3	28.80
Durability	20	Erosion	40	3	24.00
		Climate	30	2	12.00
		Chemicals	15	3	9.00
		Biologic	15	2	6.00
Construction	18	Duration	40	3	21.60
		Availability	20	3	10.80
		Quality control	40	3	21.60
Maintenance	13	Monitoring	40	2	10.40
		Duration	20	2	5.20
		Replacement	40	2	10.40
Environment	12	Pollution	40	2	9.60
		Impact	50	2	12.00
		geometry/colour	10	1	1.20
Human Factors	13	Vandalism	10	2	2.60
		Social impact	60	3	23.40
		Mishaps	30	1	3.90
TOTAL					241.30

where:

X = weight of secondary criteria in %

Y = suitability of alternative in points

- Y=0 satisfies requirements almost not at all to poorly
- Y=1 satisfies requirements poorly to sufficiently
- Y=2 satisfies requirements sufficiently to reasonably
- Y=3 satisfies requirements reasonably to well

Z = weight of primary criteria in %

G.1.8.4 Evaluation

The scores are summarized in Table G.1.8.4

Table G.1.8.4 SCORE OF ALTERNATIVES I AND II

	Alternative	
	I	II
Total score	149.90	241.30

The results show that alternative II better satisfies the selection criteria considered. The final selection should be based not only on the final score in the non-monetary MCA, but also on all capitalised cost, including capital cost and cost of maintenance etc. Dividing the final score of the MCA by the total cost, gives an idea of the best "value for money".

Chapter G.2

LOWER MEGHNA

MEGHNA RIVER BANK PROTECTION

SHORT TERM STUDY

VOLUME V- ANNEX G

CHAPTER G.2 LOWER MEGHNA

CONTENTS

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LIST OF FIGURES

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G.2.1. INTRODUCTION

G.2.1.1 General

The sites which have been prone to erosion, considered in this Short Term Study can be divided into two groups: the sites along the Upper Meghna and Dhaleswari River (Bhairab Bazar, Maniknagar, Meghna Roads and Highways Bridge and Munshiganj) and the sites along the Lower Meghna (Eklashpur, Chandpur and Haimchar). In Figure G.2.1.1 a layout map of the Meghna River is presented. The distinction between the Upper Meghna and Lower Meghna is evident and described in the River Geo-morphological Study, Annex B.

Eklashpur, Chandpur and Haimchar are situated on the left bank of the Lower Meghna River, downstream of the confluence of the Padma and the Upper Meghna. The erosion processes at the left bank are clearly related to the geo-morphological development of the Padma and the Lower Meghna in combination with wave attack. The Padma River is about six times larger when considering the discharge than the Meghna River and has shifted its course in northeast direction joining at present the Meghna River near Eklashpur instead of at Chandpur. As a consequence thereof the erosive force has increased at Eklashpur and may become more severe in future. The Lower Meghna has been eroding the left bank for more than a decade, showing a gradual shifting to the east, and has engulfed a vast area of land at a high erosion rate. At other locations, however, accretion has taken place.

The Consultants are proposing solutions for bank protective measures for these three sites along the Lower Meghna River. Solutions can be either river training works, bank protection works or other protective measures.

This Annex G, Chapter G.2, deals with the design of protection works at sites along of the Lower Meghna.

Final designs for short term protective measures will be submitted for Chandpur whereas pre-feasibility level designs will be presented for Eklashpur and Haimchar.

In addition possible emergency works have been studied for Chandpur. These will be discussed in Section G.3.

G.2.1.2 General design approach

To arrive at measures which will be technically and economically feasible and which will meet the functional requirements and sustainability, Consultants adopted a design approach which is presented in Chapter G.1 of this Annex. The same approach has been applied for the designs of the bank protection works for the Lower Meghna. For more details reference is made to this Annex, Chapter G.1,

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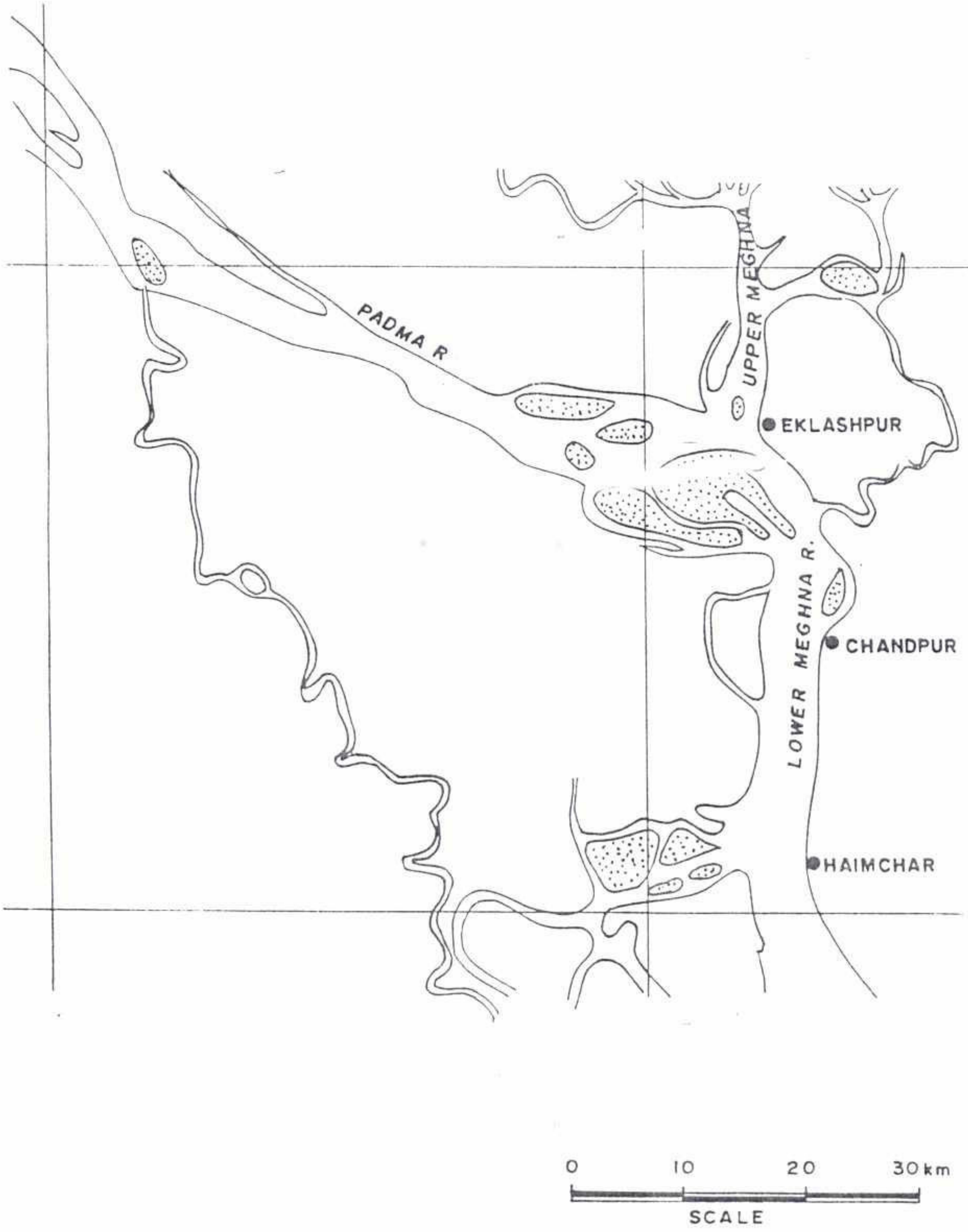

LAYOUT MAP LOWER MEGHNA | FIGURE G.2.1.1

G.2.2 ALTERNATIVE SOLUTIONS

G.2.2.1 General

In the Inception Report of this Short Term Study a priority ranking has been presented. With officials of the BWDB, FPCO and the Worldbank it was agreed to design final emergency and short term protection measures for Chandpur. For Eklashpur and Haimchar only feasibility designs for short-term protection works will be prepared.

A definitive long term solution sh... be based on the long-term geomorphological development of the river sy...em. Even mo.e, ior an understanding of the processes in the Lower Meghna one should include an analysis of the river processes in the Padma River, rather than consider the Upper Meghna alone in this respect.

The World Bank is coordinating plans for protection against erosion (Flood Protection-I Project). These measures should also be based on a Meghna Long term Strategic Plan.

The long term solution for Eklashpur, Chandpur and Haimchar may or may not require the complete training of the river system downstream of the confluence. However short term measures must be sustainable and the protection works of the sites selected should fit into such a long term solution (phased implementation program with follow up).

In the following an appreciation of the erosion problems for each site along the Lower Meghna is given. Moreover, if necessary, alternative solutions will be presented for solving the problems.

G.2.2.2 Eklashpur

G.2.2.2.1 General

Eklashpur is situated at the left bank of the Meghna River, near the confluence of Padma and Meghna (see Figure G.2.2.1). The Meghna River is under the influence of tidal action. The Meghna River has been eroding its left bank for more than a decade and has engulfed a vast area of land. The erosion is not constant. in certain ye... ...osion is not visible and in other years the erosion rate is considerable. The thalweg upstream of the confluence consists of two channels with large bars in between. The char downstream is being eroded by the channel located between the two chars.

At Eklashpur the typical cross section of the river bank shows comparatively flat slopes, 1:5 to 1:10, except for the upper part of the bank where the slopes are steeper (1:2 to 1:3) down to comparatively moderate water depths of 12 m. These slopes indicate combined action of waves and currents as the major cause of bank erosion.

During the 1988 flood the Meghna Dhonagoda Irrigation Project flood embankment was washed away. A retired embankment is at present being constructed under the BWDB Flood Rehabilitation Programme. The design of protective works for protecting the danger prone portions of the embankment, as now implemented by BWDB (ADB mission 1989, [12]) consist of a protection over a length of approximately 600 m consisting of CC-blocks and boulders. This protection should act as a falling apron apron but it is constructed above water level and it will therefore not be able to withstand erosion in future. Moreover, the proposed protection has no proper ending at the upstream part, which causes a weak point in the line of protection works.

According to the Geomorphological study Annex B, the position of a char, upstream of the confluence is being taken in by the next char within a period of approximately 16 years. It is expected that in the year 2004 the total erosion at Eklashpur w... be 700 m.

Two possible cases of geo-mor...gical development of the bank line will now be considered: a "worst case" development and a "best case" development. Both will be elaborated in the following Section.

G.2.2.2.2 Alternative solutions

Based on the above, the following aim can now be defined for Eklashpur: prevent severe scour development in front of the bank protection which can initiate bank slides or liquefaction and subsequent instability.

Accordingly the Consultants analysed the following alternative solutions for Eklashpur:

Alternative 1	Protection of existing embankment
Alternative 2	Construction of a protected retired embankment (guide bund)
Alternative 3	Overall bank protection without retirement
Alternative 4	Groyne upstream of Eklashpur

In the following sections these alternatives will be discussed in more detail.

(a) Alternative 1. Protection of existing embankment

In a "worst case of geo-morphological development" the Padma will be [illegible]ed perpendicular to the bank line at Eklashpur. According to the Geo-morphological Study, the expected bank line in this area in 2010 will have shifted some 750m. The existing embankment can be protected by a protection partly to be 'built in the dry'.

The protection starts adjacent to the BWDB protection which is now under construction. This BWDB protection work will also be integrated into the newly proposed protection. The BWDB protection is to be extended into the river. This alternative would be constructed in 3 phases (see Figure G.2.2.2).

(b) Alternative 2. Construction of a protected retired embankment (guide bund)

For the "worst case development" it is also possible to consider a retired embankment. The protection, if required, could be similar to that of Alternative 1. The protection would have the shape of a guide bund (see Figure G.2.2.3).

(c) Alternative 3. Overall bank protection without retirement

Bank protection over the whole length of the left bank. Protection with a slope of 1:3.5 and a length as shown in Figure G.2.2.4, in addition to the design as proposed by BWDB.

(d) Alternative 4. Groyne upstream of Eklashpur

A groyne placed under an angle to the embankment and a protection that would consist of gabions with boulders on a slope with a gradient of 1:3.5. Width at top is 10 m, length is 600 m and location 2.0 km upstream of Eklashpur (see Figure G.2.2.5).

G.2.2.3 Chandpur

G.2.2.3.1 General

The Chandpur Township, both Puran Bazar and Nutan Bazar, are located at the left bank of the Lower Meghna (see Figure G.2.2.6). The town is bisected by the Dakatia River. The left bank of the Lower Meghna River north and south of Chandpur has been eroding continuously for the past 20 years. During the September 1988 flood the river bank eroded over a length of about 340 m and about 40 m width.

Emergency works (temporary protection works) have been carried out in the past and are being carried out today to protect Chandpur against the very substantial erosive forces of the Lower Meghna. The

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measures taken so far mainly consisted of placing (dumping) of coarse elements (boulders, geotextile bags filled with sand, gunny bags filled with bricks, concrete blocks) on attacked and/or eroding slopes, without any regard for building up a proper filter layer. Only very recently (1990) attempts have been made to create a filter, consisting of sand filled geotextile bags, under a protective layer of concrete cubes[1].

Measures implemented in the past had one thing in common: a very substantial part of the protective elements have been placed on eroding, and thus, oversteep slopes. This by itself is not surprising, because:

- local contractors do not have the resources (equipment, experience) to create slopes under water which are sufficiently stable.

- the lead time allowed is too short for preparation of working areas where, for instance, proper slope protection mattresses can be made or materials can be stockpiled.

- the very nature of the temporary emergency works does not allow for the time consuming process of ICB; so LCB is applied;

The consequence of the above is that the emergency protective measures have a very limited life time (no or not properly functioning filter; oversteep slopes with subsequent slides). Therefore almost every year new emergency measures had and have to be taken.

Four basic aspects should, under any circumstances, be observed when proposing design concepts for a durable protection of Chandpur Town:

- slopes should be stable under various conditions, including earthquakes and ground water flow induced by rapidly falling river levels;

- protective elements should be able to withstand the erosive forces of the currents and, to a lesser degree, waves;

- a filter, in the form of a geotextile or a granular filter, should prevent the migration of soil from under the protective layer(s);

- provisions should be made for scour depths in excess of the prevailing river depths at the time of construction.

For Chandpur Town application of a new protective layer on the existing slopes would not satisfy all these criteria, in particular the slope stability criterion cannot be satisfied. Apart from this aspect the highly irregular slopes would exclude the possibility to apply slope protection mattresses which would incorporate geotextiles. A proper filter would thus have to be made up solely of granular materials. The quantity of granular materials would then become extremely high[2].

Major parts of the upper areas of the bank along the township have been protected by a boulder revetment, which is not in a good condition. Some of the revetments have been made in an isolated manner, without a general plan. In 1990, two protection works in front of Nutan Bazar and Puran Bazar have been finalised as per National Committee recommendation. They consist of CC-blocks placed on geotextile sand bags, the latter are supposed to act as filter.

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[1] In view of the insufficiently accurate method of placing and fill in the bags with too fine sand, it is doubtful that the layer of geotextile bags is functioning as intended.

[2] Assuming for the moment that rock with D_{5-} = 0.35m would satisfy the current resistance criteria, the total thickness of rock on a filter fabric would be in the order of D_{50} 2 = 0.70m, whereas an overall graded filter should have a thickness of a corresponding D_{65} * 5 = 0.35 * 5 = 1.75m.

During the 1990 flood part of the Nutan Bazar area was devoured by the Meghna and severe erosion took place at Puran Bazar. A strong current is directly striking Chandpur Town, causing scour depths in front of the revetments up to 55 m. Moreover, wave induced erosion takes place during high water stages.

After the Cyclone of April 1991 the situation worsened. Large areas of Nutan Bazar and Puran Bazar have been eroded and this process still continues; the Mosque and the Railway Station are at present in severe danger and the site of the terminal of Jamuna Oil has already been abandoned.

G.2.2.3.2 Alternative solutions

Based on the above, the following two aims can now be defined for Chandpur Town:

i) prevent geotechnical instability of the land areas near the Mosque and Railway station;

ii) accommodate the effects of severe scour development in front of the bank protection works of Nutan Bazar and Puran Bazar which can initiate bank slides.

There are essentially three methods to arrive at an acceptable slope gradient for a protective embankment along the perimeter of Chandpur Town[3]:

- Cut and fill: a minimum quantity of earth is to be moved. For Chandpur there do not seem to be reasonable and affordable methods available to cut into the existing slopes (very large depths and presence of a large quantity of coarse to very course materials, such as concrete blocks originating from previous emergency works and remnants of collapsed buildings). Placing of gunny bags filled with earth (sand) seems to be one of the few, if not the only possible method to fill out the slopes to the required lines and levels; however, the interface between the oversteep slopes and the gunny bag fill, would remain a week point: there will be large voids between the bags, so that the gunny bag fill must be expected to be highly deformable. Accordingly, the fill may not contribute to achieving the required stability of the slope.

- Cut only: a new slope, to be cut at the required gradient, on which the slope protection can be applied. To arrive at such a slope a very substantial part of Chandpur would have to be sacrified. Moreover, cutter dredging would have to be carried out to very large depths (say 50m or more), which is beyond the reach of standard equipment available in the market.

- Fill only: a new filled embankment having the required slope gradient. The only reasonable method would be by means of hydraulic filling. The gradient of an underwater slope which can be achieved by solely discharging hydraulic fill under water will probably be in the region of 1:10. If such a method would be applied then one would have to re-dredge later to the required gradient of 1:3.5 (or so). The large depths at Chandpur will make such an exercise extremely costly. Therefore it is more attractive to arrive at the required slope gradient by the construction of successive layers of containment bunds, consisting of coarse granular materials. A containment bund could be three metres high and would serve to contain the hydraulic fill. The new embankment should thus be built up in layers having a thickness of three metres. Thicker layers would require more coarse granular material to be placed in each containment bund. (see Figure G.2.2.7)

3 Other schemes than embankments, such as groynes, are not attractive in the existing situation, as will be discussed elsewhere.

The picture which emerges from the above mentioned possibilities is that a new embankment made up of hydraulic fill, making use of containment bunds of coarse materials, is the only reasonable option for a durable protection of Chandpur Town.

Bearing in mind these considerations Consultants analyzed the following alternative solutions for Chandpur Town:

Alternative 1	Protective layers on existing slopes at Nutan Bazar and Puran Bazar improving present slopes by cut and fill;
Alternative 2	Advanced protection in front of Nutan Bazar and Puran Bazar;
Alternative 3	Series of 2 groynes in front of Nutan Bazar;
Alternative 4	Groyne upstream of Chandpur;
Alternative 5	Submerged sa[illegible]usages in upstream direction of Chandpur.

In the following sections these alternatives will be discussed in more detail.

(a) <u>Alternative 1. Protective layers on existing slopes at Nutan Bazar and Puran Bazar improving present slopes by cut and fill</u>

To prevent further erosion in some specific areas protective layers are proposed on the existing banks improving slopes to 1:3.5. Some filling and excavation is required. Slope protection can consist of CC-blocks, rock or sack gabions.

(b) <u>Alternative 2. Advanced protection in front of Nutan Bazar and Puran Bazar</u>

As part of an overall bank protection it is possible to consider a so called advanced[4] protection by formation of a strip of reclaimed land in front of the existing town protection; Nutan Bazar and Puran Bazar or only in front of Nutan Bazar (see Figure G.2.2.8 and G.2.2.9). Backfilling of large irregularities is required by hydraulic filling or dumping of gunny bags filled with soil.

(c) <u>Alternative 3. Series of 2 groynes in front of Nutan Bazar</u>

A series of 2 groynes in front of Nutan Bazar with lengths of 75 m at the top and of approximately 250m. at the river bed. Possible sedimentation will occur just downstream of these groynes, thus creating an area on which Alternative 2 can easily be constructed reducing the volume of filling. This alternative can also be considered as emergency measure (see Figure G.2.2.10).

(d) <u>Alternative 4. Groyne upstream of Chandpur</u>

A large groyne 2 or 3 km upstream of Chandpur having a length of 600 m or 800 m connected by a closure dam to the existing embankment. The angle with the bank line can vary (see Figure G.2.2.11 and G.2.2.12).

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[4] 'Advanced' is used here as opposite of 'retired', it does not necessarily imply the use of advanced techniques.

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(e) Alternative 5. Submerged sand sausages in upstream direction upstream of Chandpur

Submerged sand sausages in upstream direction upstream of Chandpur in order to deviate the flow from Chandpur Town. Sausages would have a diameter of 7 up to 10m. Location 3 km upstream of Chandpur. Length 800m. Spacing 100m or 50m.

G.2.2.4 Haimchar

G.2.2.4.1 General

Haimchar is situated about 20 km south of Chandpur Town at the left bank of the Lower Meghna River (see Figure G.2.2.13). The river width is about 10 km and since 1929 an ongoing bank erosion of the left bank has been reported. At present the erosion rate is alarming and the bank has moved about 200 m in one year. Parts of the flood embankment of the Chandpur Irrigation Project have been engulfed by the river several times. In 1989 BWDB built already a retired embankment to protect the irrigation project area.

G.2.2.4.2 Alternative solutions

Protection works at Haimchar should:

i) prevent scour development in front of the bank line;

ii) accommodate the effects of scour development due to wave attack.

Accordingly, Consultants analyzed and identified the following alternative solutions for Haimchar:

Alternative 1 Protection of partly retired embankment;

Alternative 2 Protection of existing embankment;

Alternative 3 Groyne(s) upstream of Haimchar;

These alternatives will be discussed in more detail below:

(a) Alternative 1. Protection of partly retired embankment

Protection of a partly existing embankment at the area where severe erosion has occurred. This protection would have the shape of a guide bund. In some areas it is necessary to replace the embankment. Such a protection is to be partly built in the dry by excavation of a trench in front of the embankment (see Figure G.2.2.14 for general layout). For this basic alternative several implementation schemes are possible, varying both in time and in location.

(b) Alternative 2. Protection of existing embankment

Protection of a existing embankment at the area where severe erosion has occurred. Contrary to Alternative 1 for this alternative the protection follows the existing embankment. Such a protection is to be partly built in the dry by excavation of a trench in front of the existing embankment (see Figure G.2.2.15 for general layout). For this alternative also several implementation schemes are possible.

(c) Alternative 3. Groyne(s) upstream of Haimchar

In order to deviate the flow from Haimchar a groyne can be considered upstream of Haimchar.

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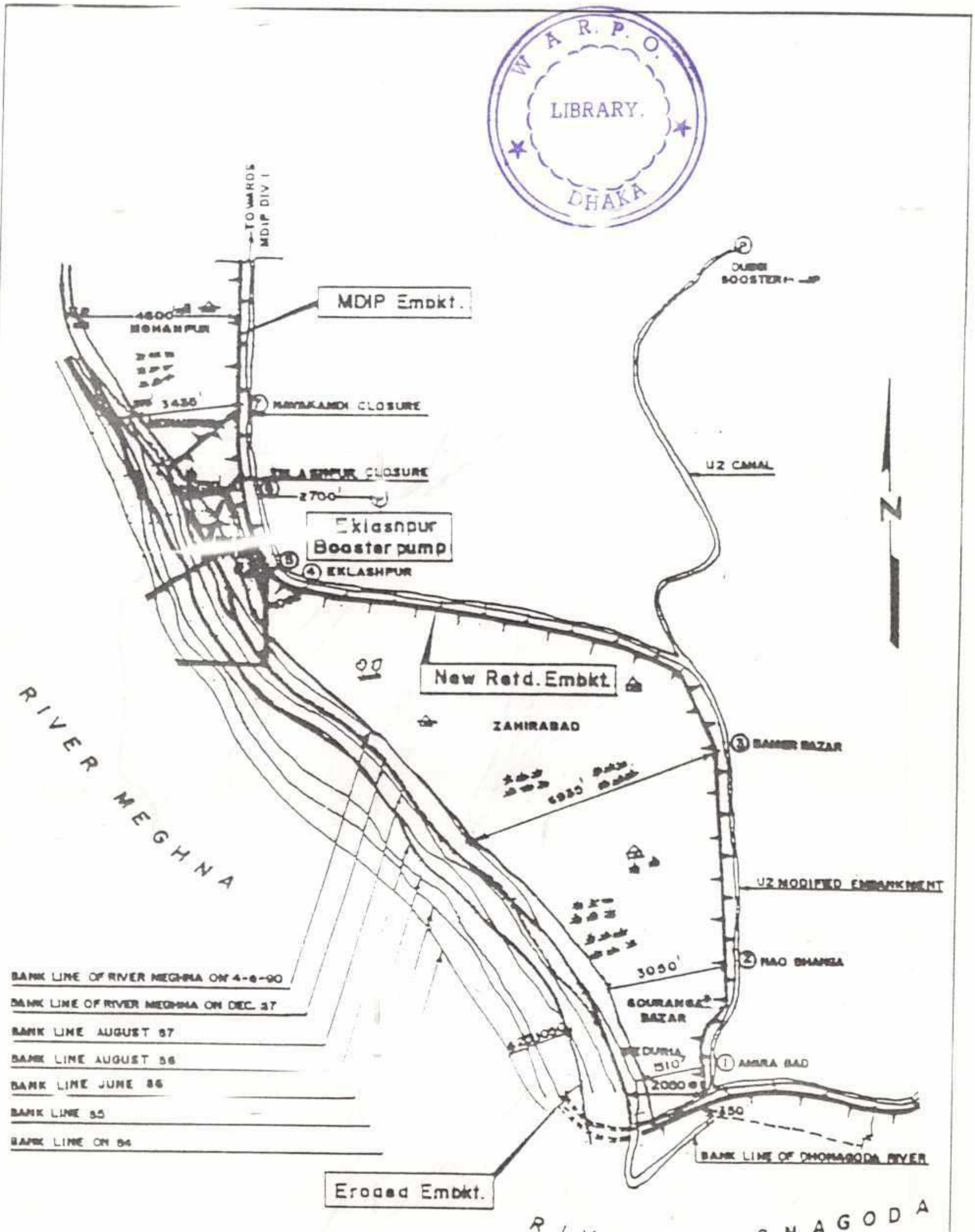

LAYOUT PLAN EKLASHPUR

FIGURE. G. 2. 2. 1

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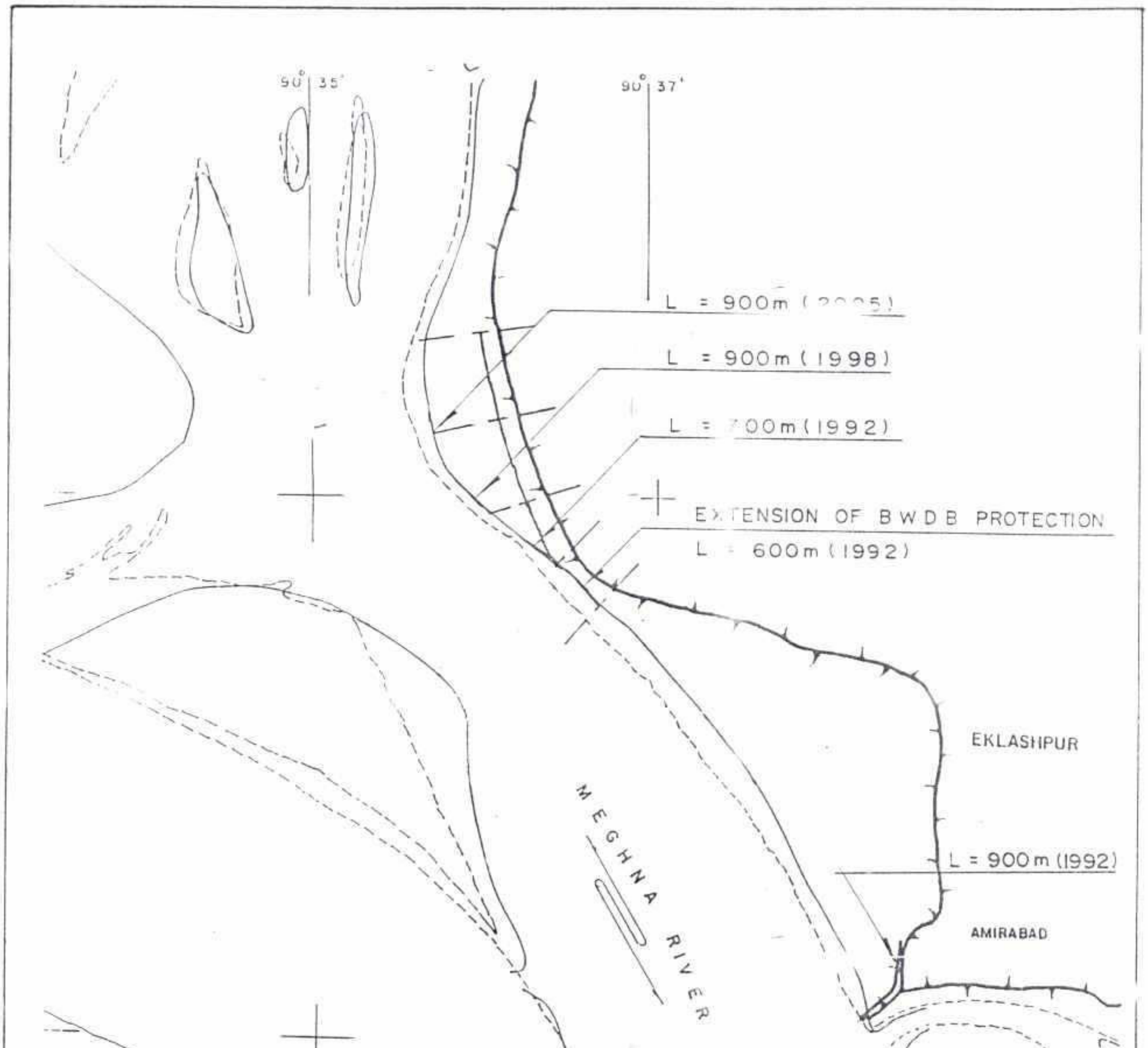
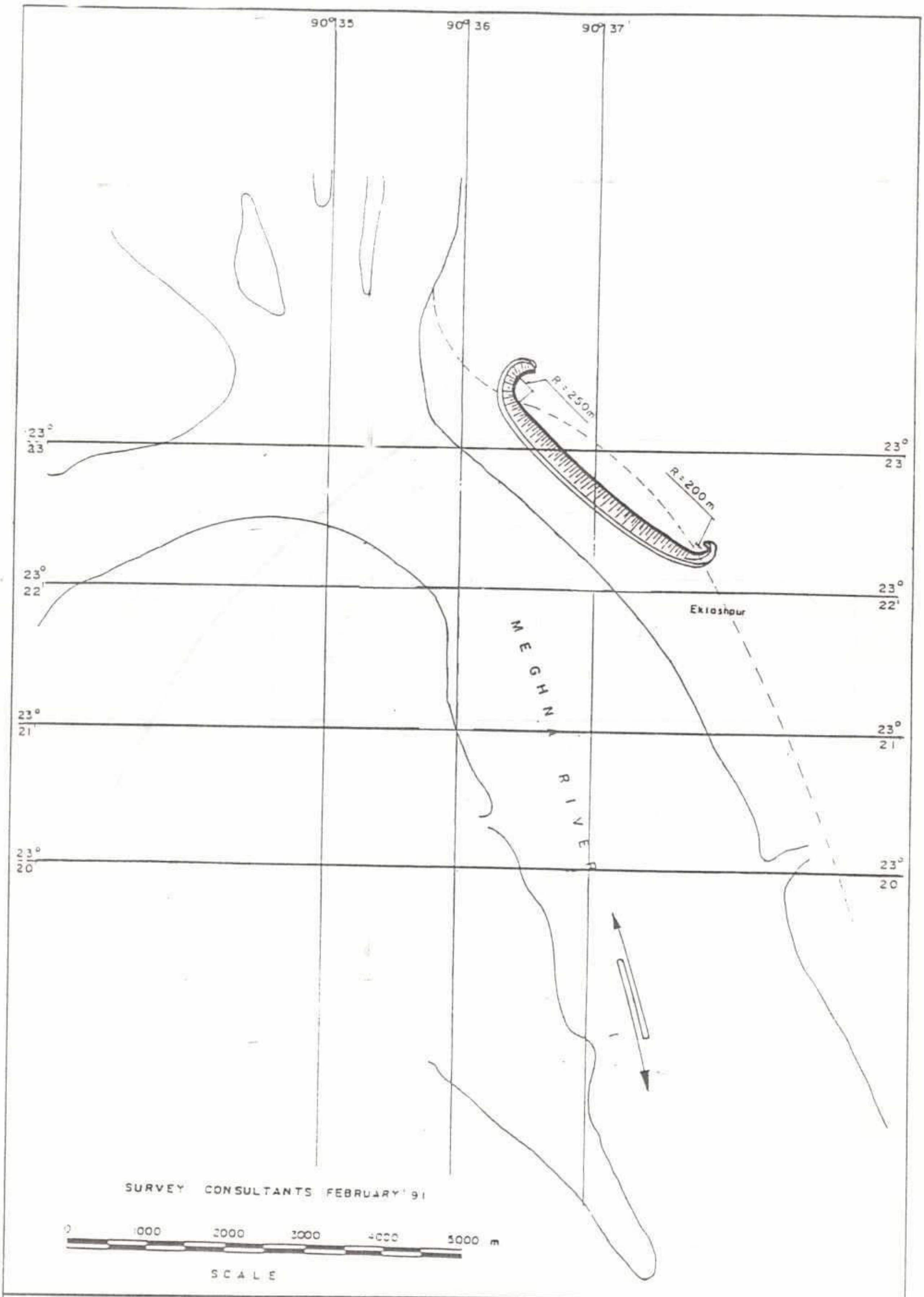
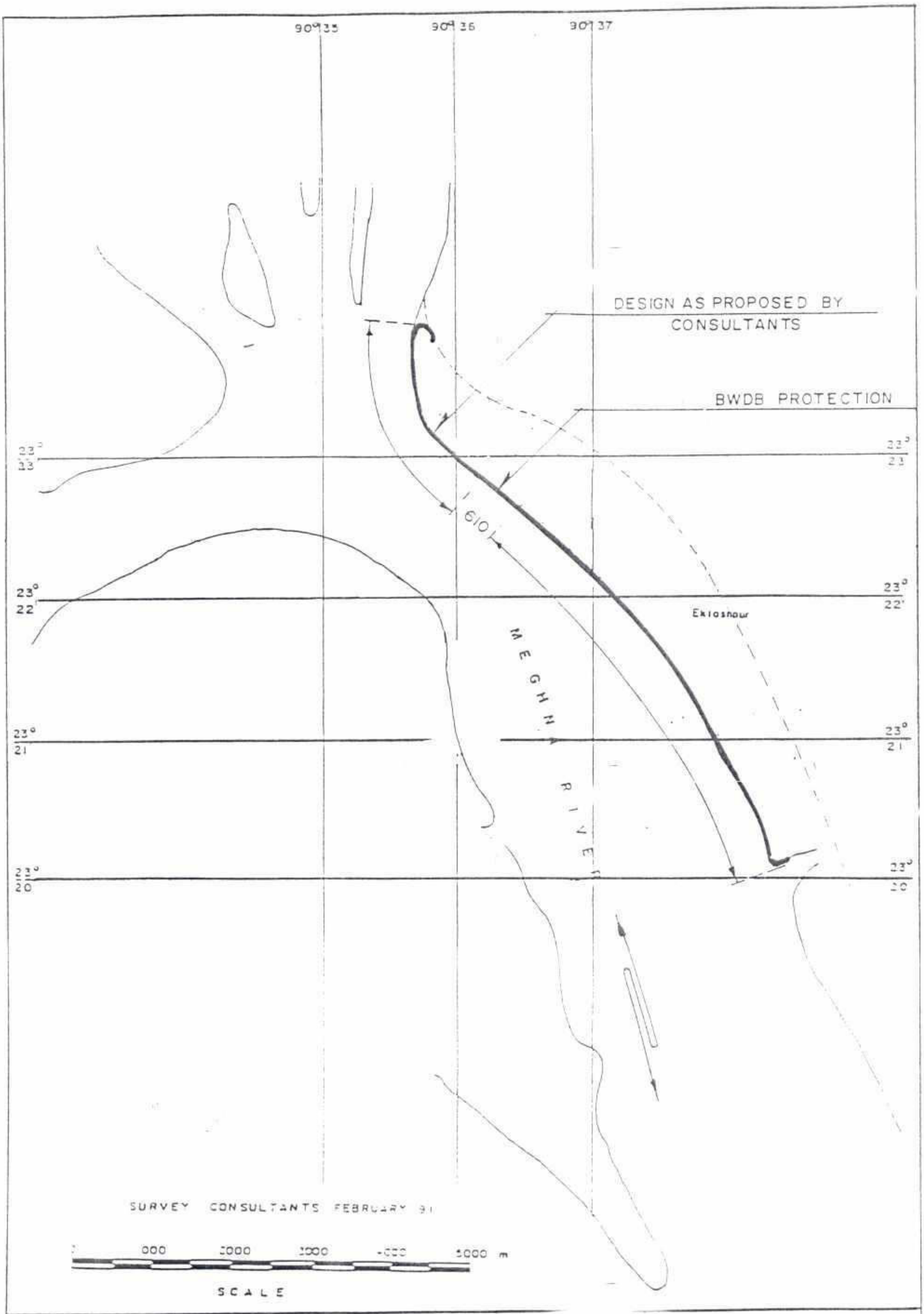


PROTECTION OF EXISTING EMBANKMENT EKLASHPUR	FIGURE G. 2. 2. 2



RETIRED EMBANKMENT EKLASHPUR

FIGURE G.2.2.3



PROTECTION EXISTING BANK LINE EKLASHPUR

FIGURE G.2.2. 4

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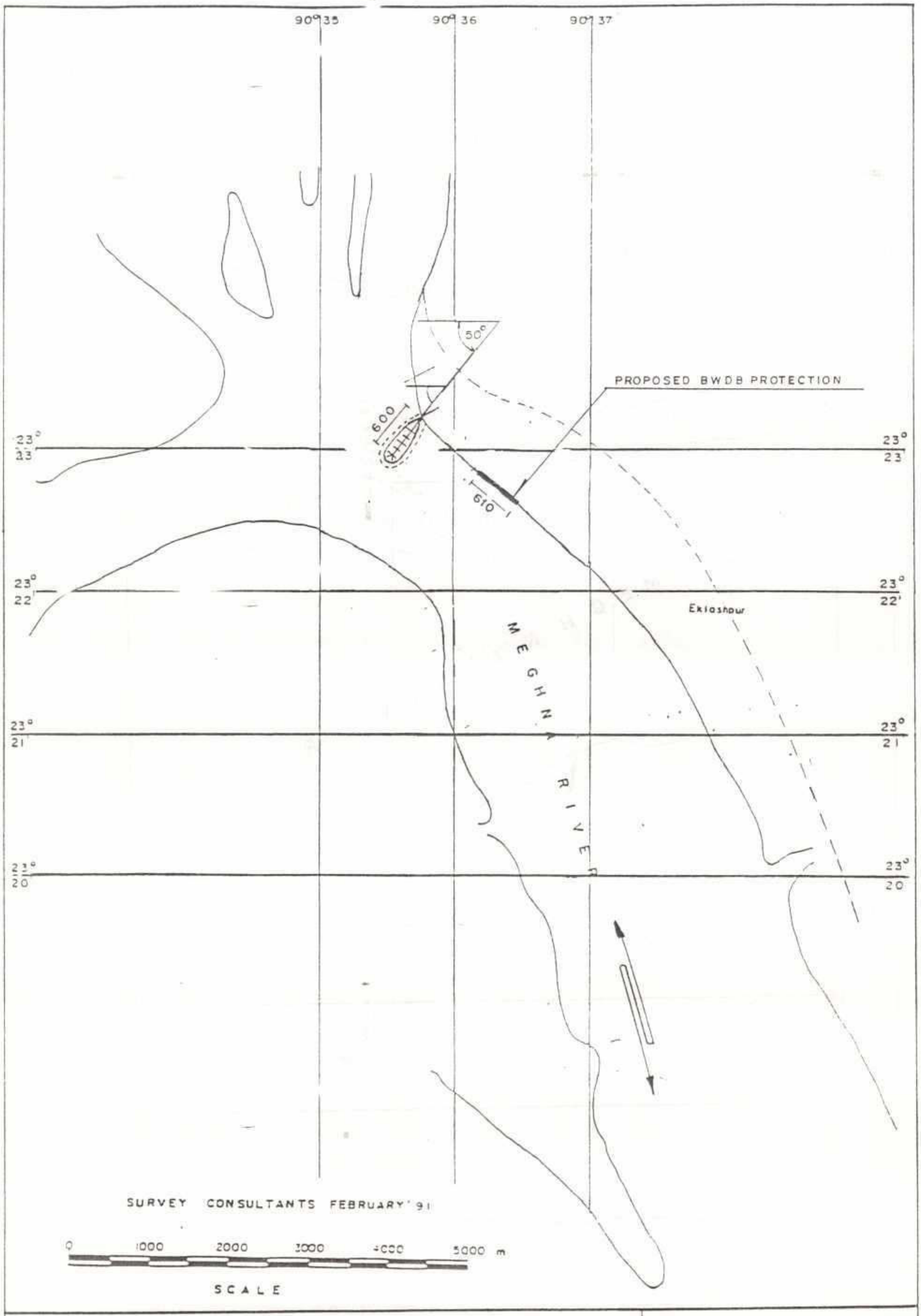

GROYNE UPSTREAM OF EKLASHPUR	FIGURE G.2.2.5

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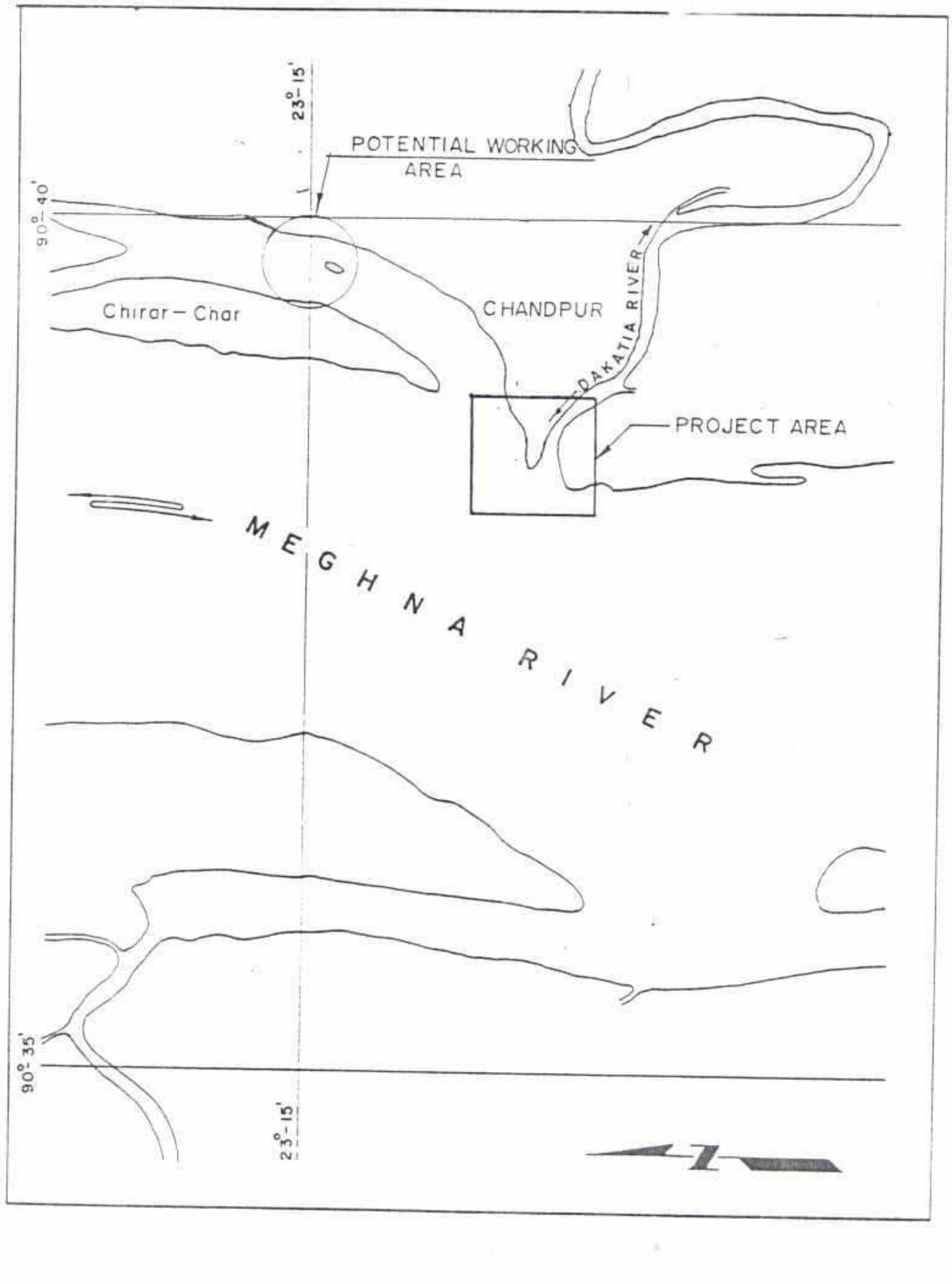
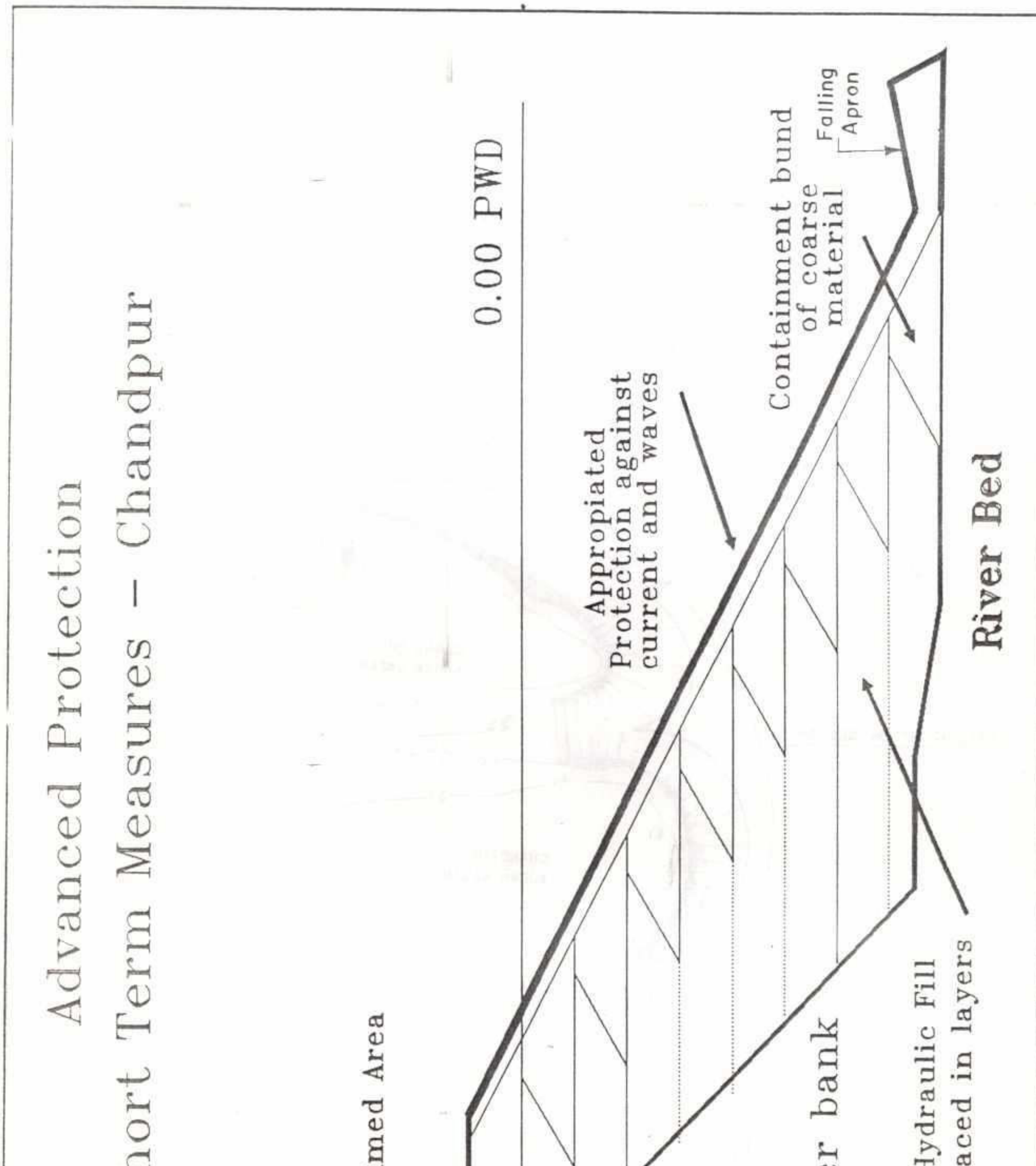


LAYOUT PLAN CHANDPUR TOWN	FIGURE G.2.2.6



CONCEPT OF ADVANCED PROTECTION CHANDPUR	FIGURE G.2.2.7

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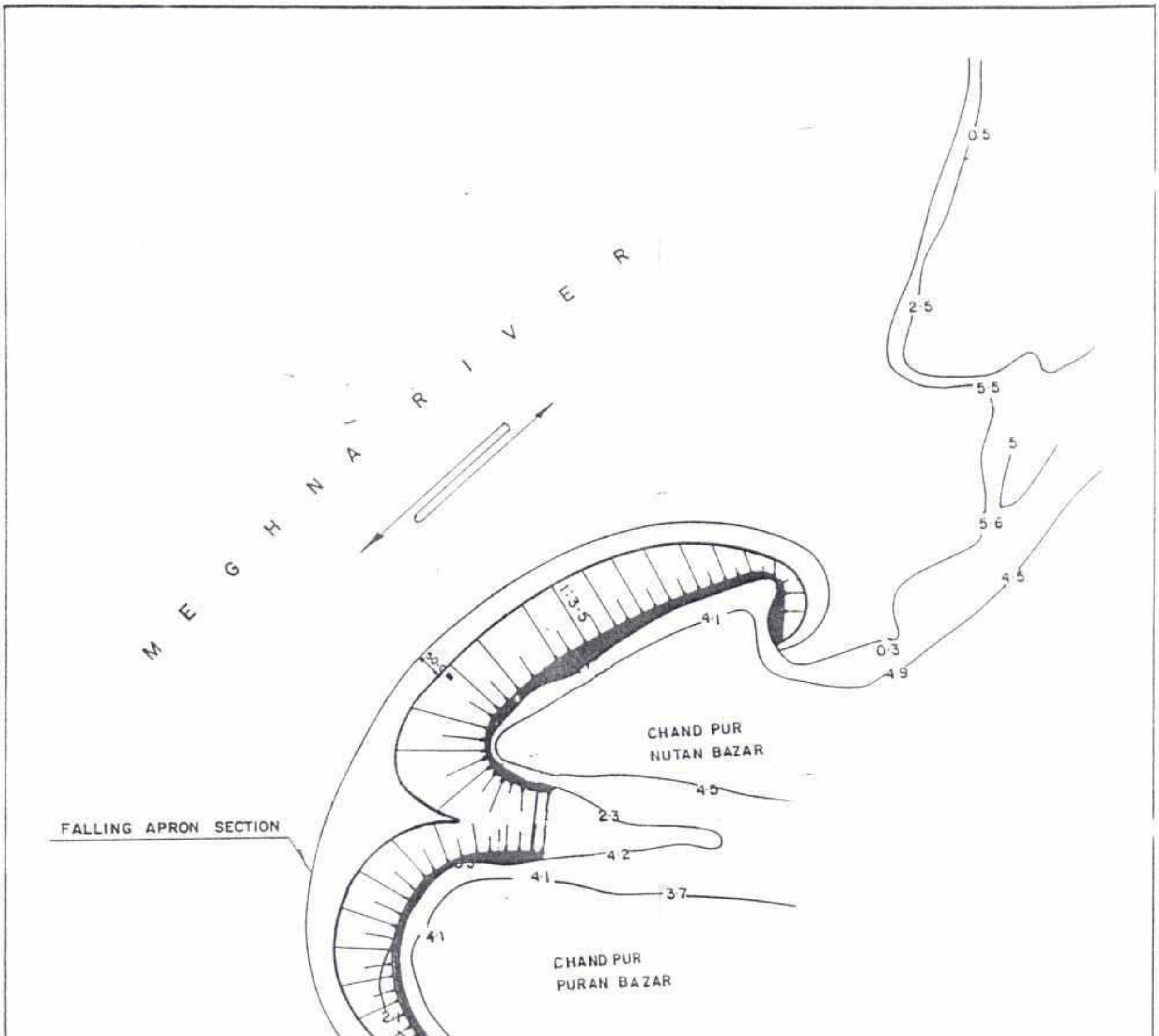

ADVANCED PROTECTION NUTAN BAZAR AND PURAN BAZAR	FIGURE G.2.2.8

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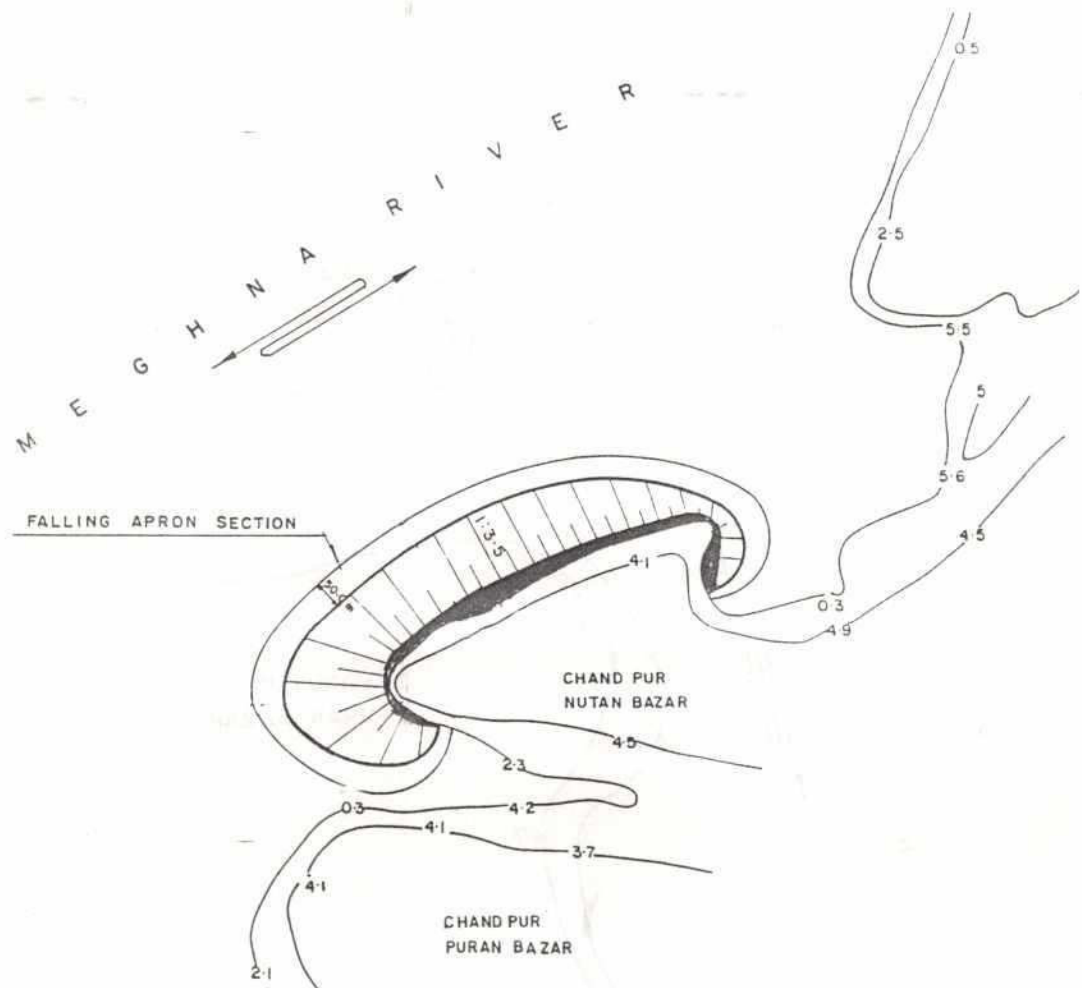

ADVANCED PROTECTION NUTAN BAZAR	FIGURE G.2.2.9

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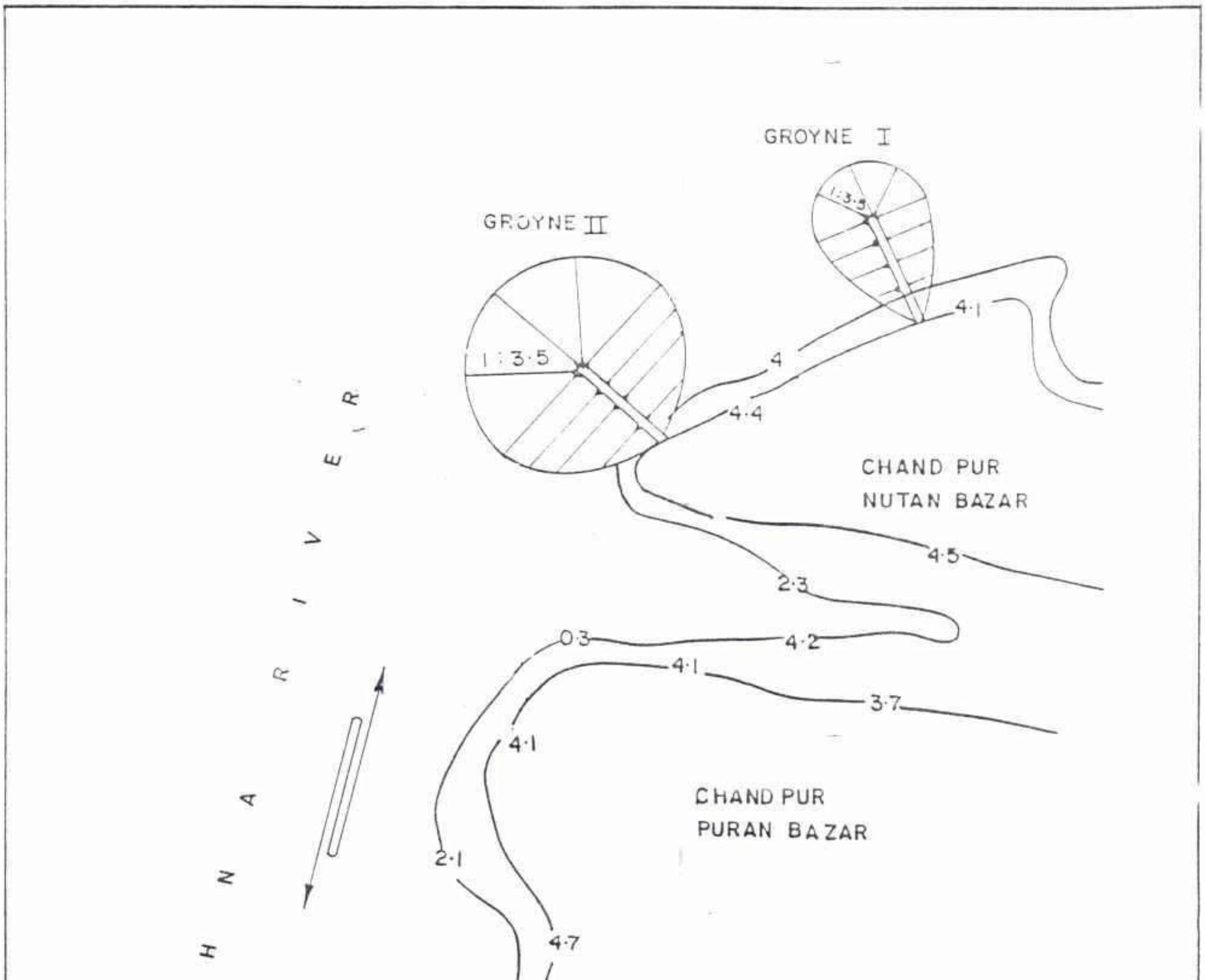

GROYNES IN FRONT OF NUTAN BAZAR

FIGURE G.2.2.10

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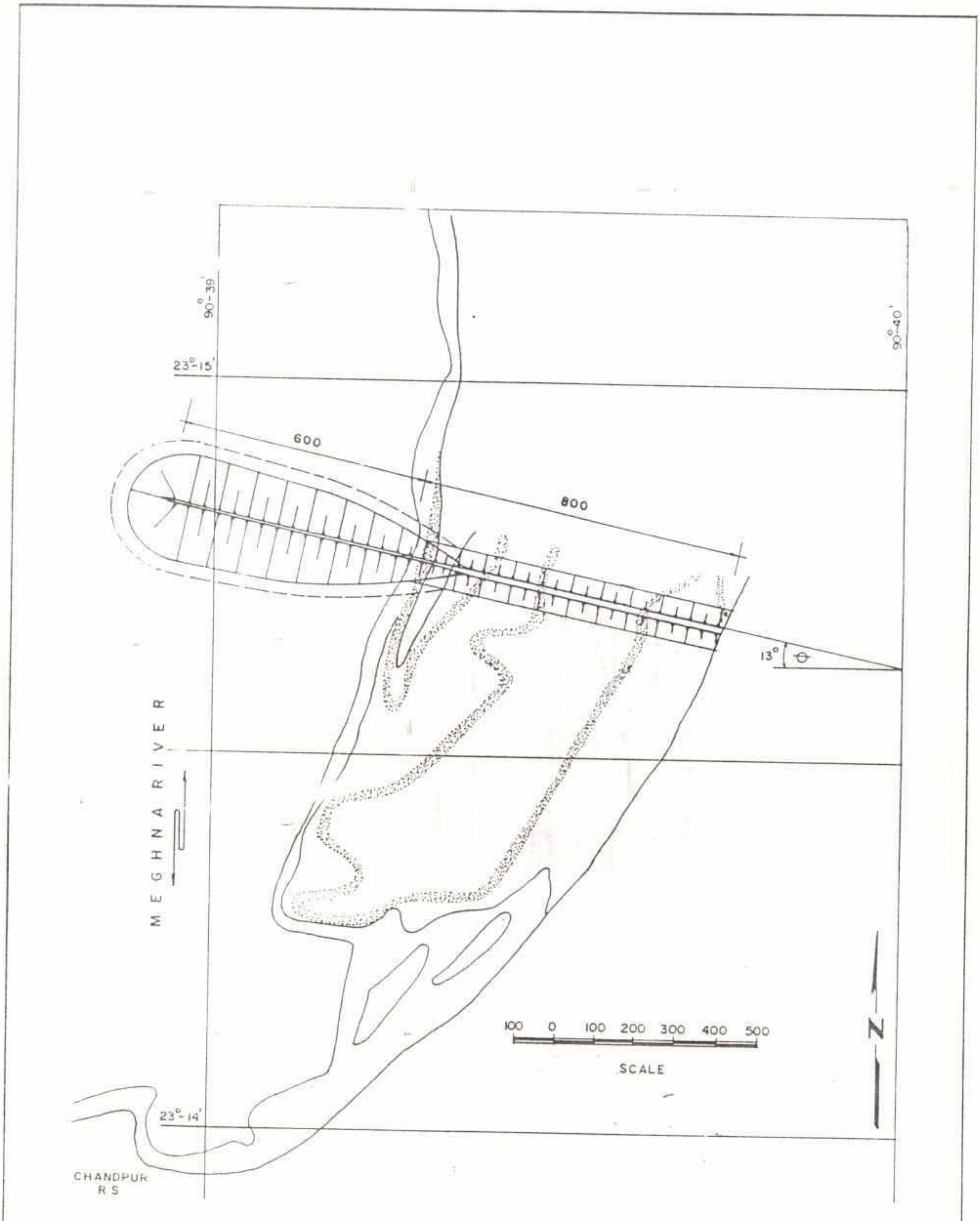
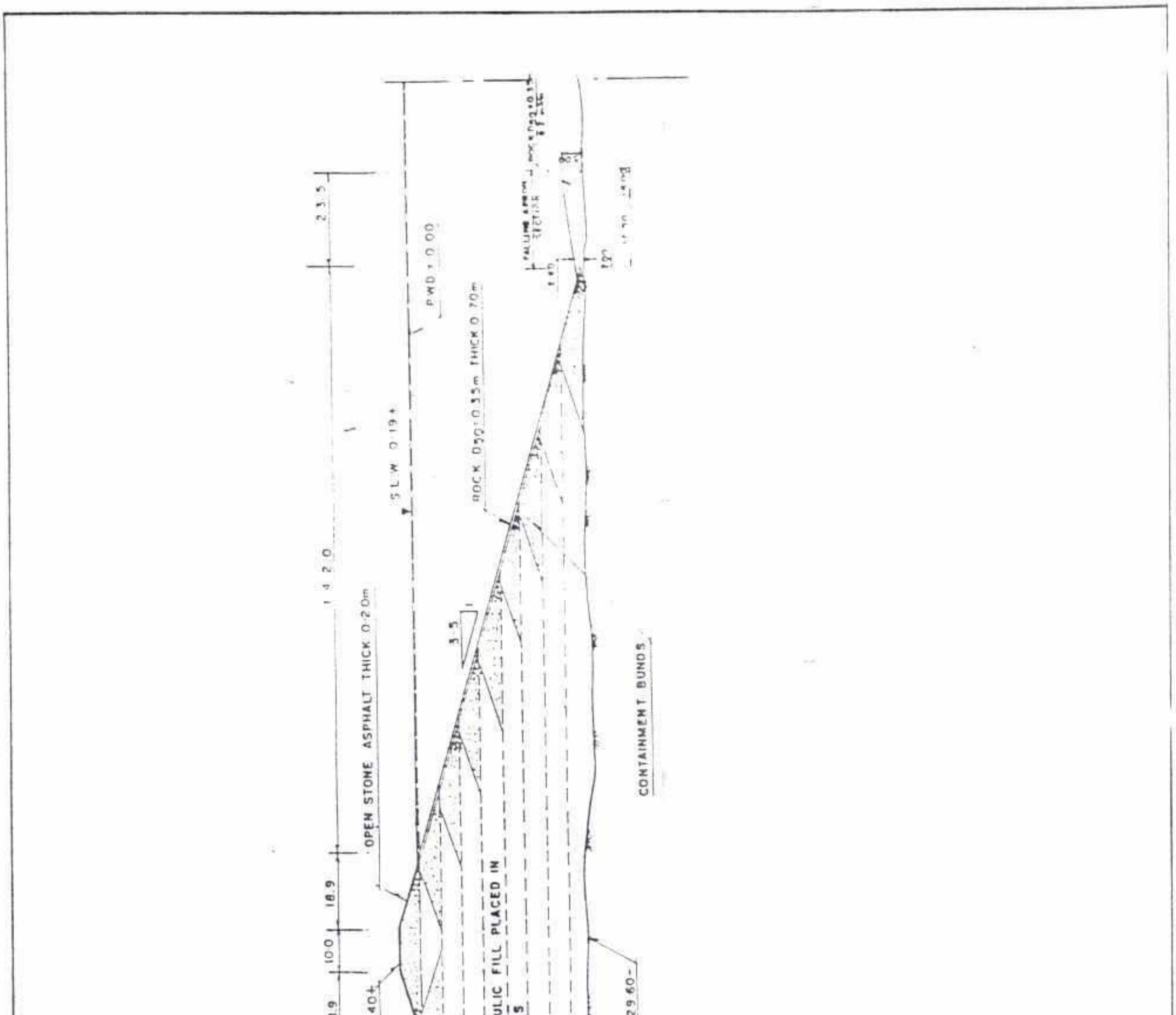


GROYNE UPSTREAM OF NUTAN BAZAR

FIGURE G. 2. 2. 11



TYPICAL CROSS SECTION
GROYNE UPSTREAM OF CHANDPUR

FIGURE NO. G. 2.2.12

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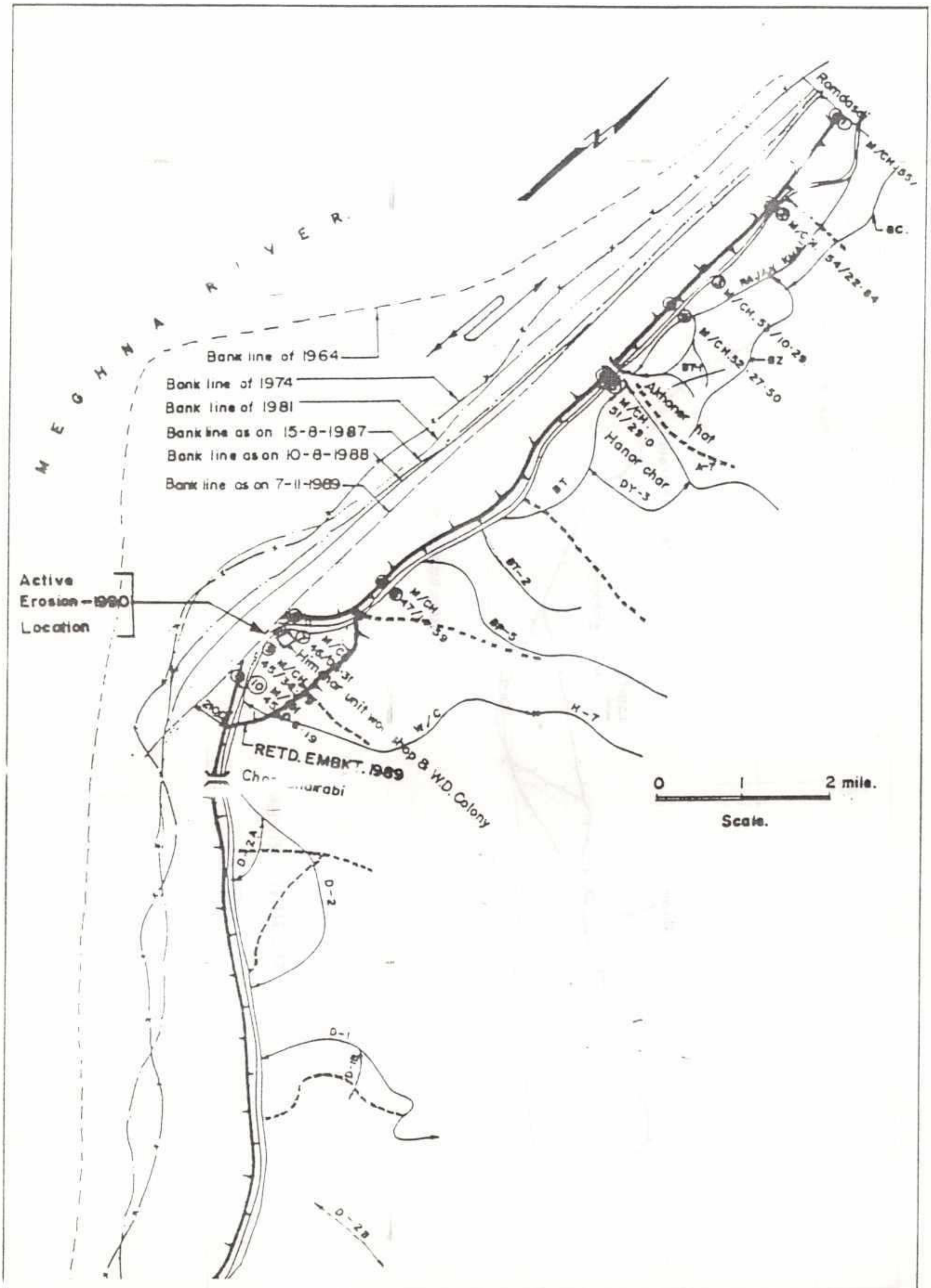
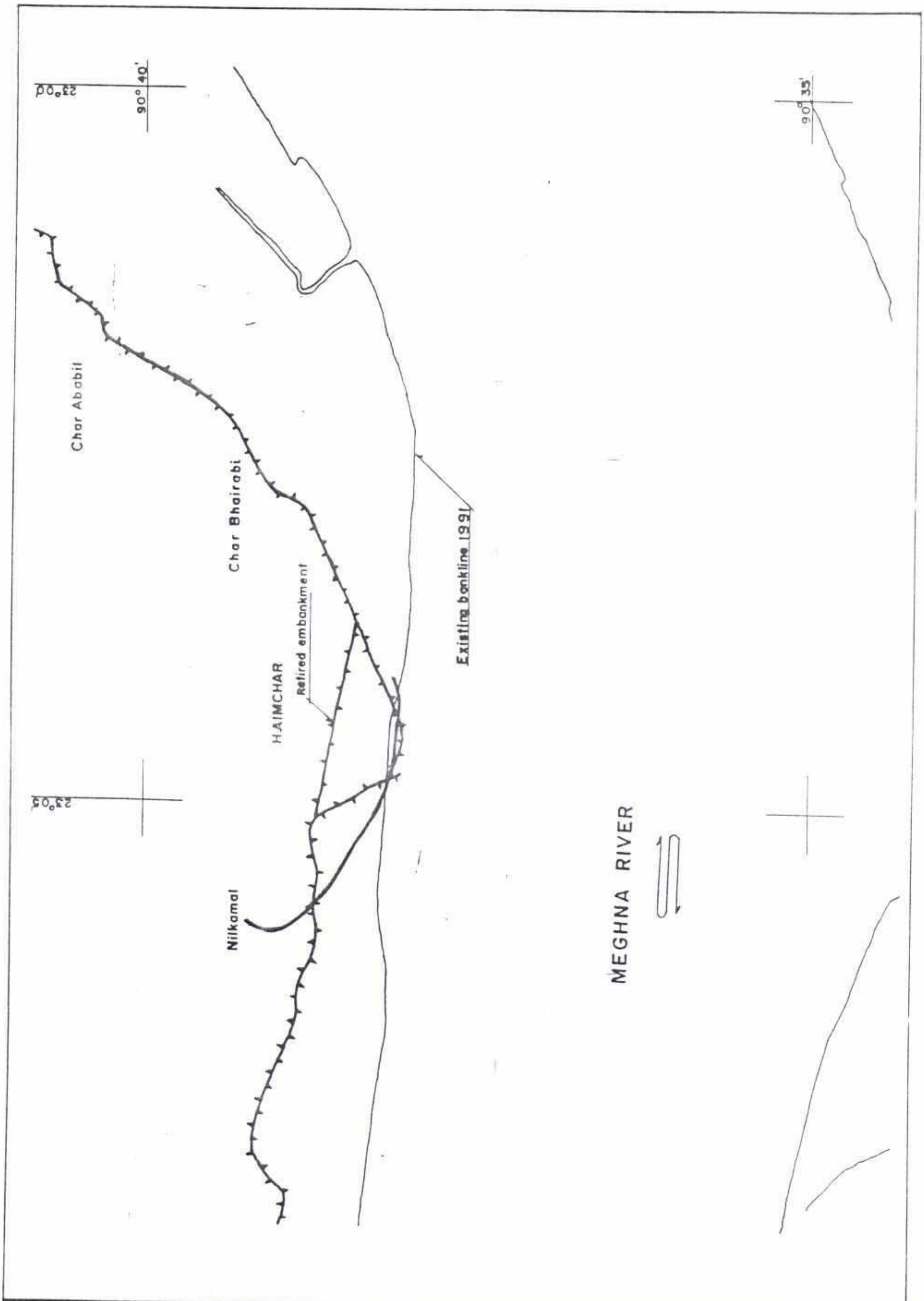
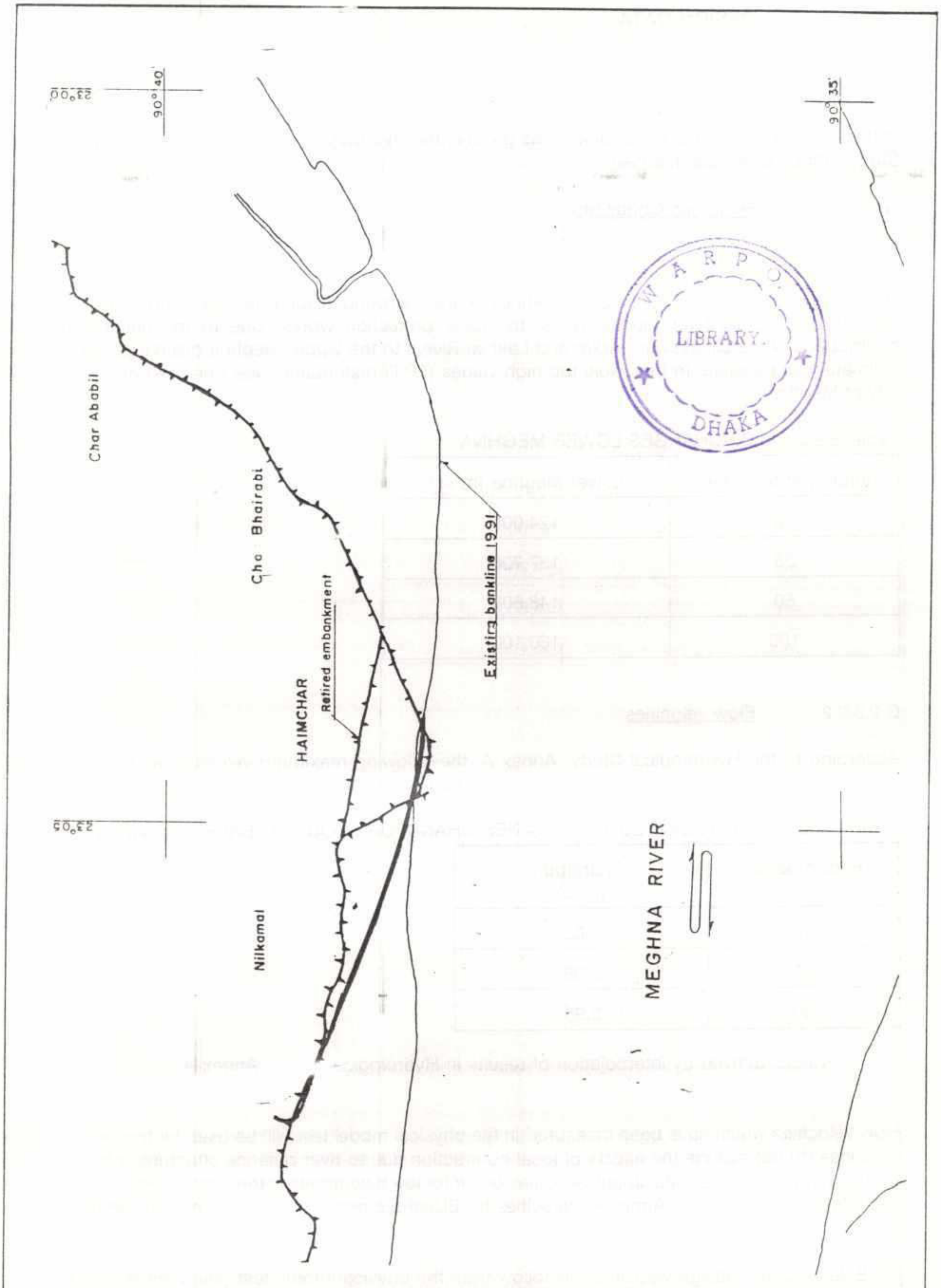


LAYOUT PLAN HAIMCHAR	FIGURE G.2.2.13



PROTECTION OF PARTLY RETIRED EMBANKMENT HAIMCHAR	FIGURE . G.2.2.14



PROTECTION OF EXISTING EMBANKMENT HAIMCHAR	FIGURE G.2.2.15

G.2.3 DESIGN DATA

G.2.3.1 General

In this Section the boundary conditions, as given in the Hydrological Study, Annex A and Geotechnical Study, Annex C are summarised.

G.2.3.2 Hydraulic Conditions

G.2.3.2.1 Discharges

According to the Hydrological Study, Annex A the following discharges and corresponding return periods can be used for the design of the bank protection works. Due to the uncertainty in the contribution of the Dhaleswari, Aram and Lakhya Rivers to the Upper Meghna discharge, only a rough estimate was possible. In the Table the high values (LP-III distribution, see Annex A) are given for the Lower Meghna.

Table G.2.3.1 DISCHARGES LOWER MEGHNA

Return period (years)	Lower Meghna (m^3/s)
10	124,000
25	137,700
50	148,600
100	160,100

G.2.3.2.2 Flow velocities

According to the Hydrological Study, Annex A, the following maximum velocities and corresponding return periods can be used for the design of the bank protection works

Table G.2.3.2 MAXIMUM FLOW VELOCITIES CHANDPUR CLOSE TO BANK IN A VERTICAL

Return period (years)	Chandpur (m/s)
10	3.02
25	2.98
50	2.95

*) values derived by interpolation of results in Hydrological Study, Annex A.

Flow velocities which have been measured in the physical model test will be used for the design. These velocities do not include the effects of local contraction due to river defence structures which will lead to even higher velocities. Maximum velocities occur for low discharges in the Lower Meghna. For a more elaborated discussion see Annex A. Velocities for Eklashpur and Haimchar are not presented in Annex A.

For Eklashpur the design velocities will follow from the physical model test (see Section G.2.4.2.3).

The design flow velocity for Haimchar will follow from the Mathematical Model Studies, Annex E. In

Annex E it is stated that in front of Haimchar flow velocities of 1.9 m/s occur. The latter value will be used for design purposes.

G.2.3.2.3 Water Levels

Waterlevels have been retrieved from BWDB sources and year books and presented in the Hydrological Study, Annex A. According to the Hydrological Study the following waterlevels and corresponding return periods can be used for the design of the bank protection works.

Table G.2.3.3 MAXIMUM WATERLEVELS LOWER MEGHNA

Return period (years)	Eklashpur (m +PWD)	Chandpur (m +PWD)	Haimchar (m +PWD)
10	5.49	5.08	4.78
25	5.67	5.20	4.87
50	5.80	5.29	4.94
100	5.93	5.37	5.00

If values were not presented in Annex A waterlevels have been interpolated between values presented in Annex A.

Some other important levels are Standard High Water (SHW), which is the waterlevel exceeded during 18 days per year, and Standard Low Water (SLW), which is the waterlevel not exceeded during 18 days per year. In the following Table these values are listed. These water levels have been used for the design.

Table G.2.3.4 CHARACTERISTIC WATERLEVELS LOWER MEGHNA

	Eklashpur	Chandpur	Haimchar*)
SHW (m +PWD)	6.04	5.16	4.79
SLW (m +PWD)	0.55	0.19	0.73

*) values for Nilkamal have been adopted

G.2.3.2.4 Waves

Waves at the site would either be generated by wind or by ships. Data on wind waves were not available. Based on wind data from the meteorological station Chandpur, predictions of the wind waves have been made. Waves generated by ships have also been considered.

In the following Table some characteristics are presented of the wind and wind generated waves.

Table G.2.3.5 WIND AND WAVE CHARACTERISTICS LOWER MEGHNA

Return period (years)	Wind velocity (m/s)	Wave height Hs (m)	Wave period Ts (sec)
1	8.0	0.42	2.55
10	16.0	0.96	3.71
100	29.0	1.25	4.18

The wave heights have been calculated by applying Bretschneiders formula.

G.2.3.2.5 Sediment and water characteristics

Bed samples which have been taken in the month of February 1991 were analyzed at the RRI in Faridpur. In Table G.2.3.6 some values are listed. In Figure G.2.3.1 a layout map with cross sections is shown were bed samples were taken.

Table G.2.3.6 SEDIMENT CHARACTERISTICS

	Chainage (km)	D_{16} (mm)	D_{50} (mm)	D_{84} (mm)
Left	105.9	.02	.074	.1131
	111.2	.12	.18	.2395
	118.8	.025	.09	.1835
	126.5	.06	.12	.188
	134.5	.03	.035	.17525
	142.4	.008	.035	.06815
	150.5	.004	.0028	.04292
Middle	105.9	.02	.09	.1155
	111.2	.12	.2	.3275
	118.8	.074	.149	.19235
	126.5	.015	.06	.111
	134.5	.055	.12	.2305
	142.4	.08	.15	.3268
	150.5	.01	.055	.10175
Right	111.2	.055	.14	.208
	118.8	.02	.06	.111
	126.5	.074	.15	.235
	134.5	.01	.04	.0825
	142.4	.12	.18	.384
	150.5	.05	.12	.205

For Chandpur the following characteristic grain size diameters have been derived from these reports:

$$\mu(D_{50}) = 0.070\ mm \qquad \sigma(D_{50}) = 0.052\ mm$$

$$\mu(D_{90}) = 0.147\ mm \qquad \sigma(D_{90}) = 0.082\ mm$$

Similarly for Eklashpur the following grain size diameters have been derived:

$$\mu(D_{50}) = 0.092\ mm \qquad \sigma(D_{50}) = 0.056\ mm$$

$$\mu(D_{90}) = 0.183\ mm \qquad \sigma(D_{90}) = 0.092\ mm$$

For obtaining sand for hydraulic filling operations, the coarsest possible material should be used.

G.2.3.3 Geotechnical characteristics

The Geotechnical Study, Annex C presents a review of all data, arriving at specific layer classification and design parameters.

The following determining characteristic parameters are derived from it for Chandpur:

- $D_{90} = 0.09\ mm$
- $k_{soil} = 5.961\ 10^{-7}\ m/s$ (*Nutan Bazar*)
- $k_{soil} = 1.602\ 10^{-7}\ m/s$ (*Puran Bazar*)

For design purposes it is assumed that these parameters are also applicable for the sites of Eklashpur and Haimchar.

G.2.3.4 Climatology

For information and typical data on the climatology in Bangladesh reference is made to Chapter G.1, Section G.1.3.2.1.

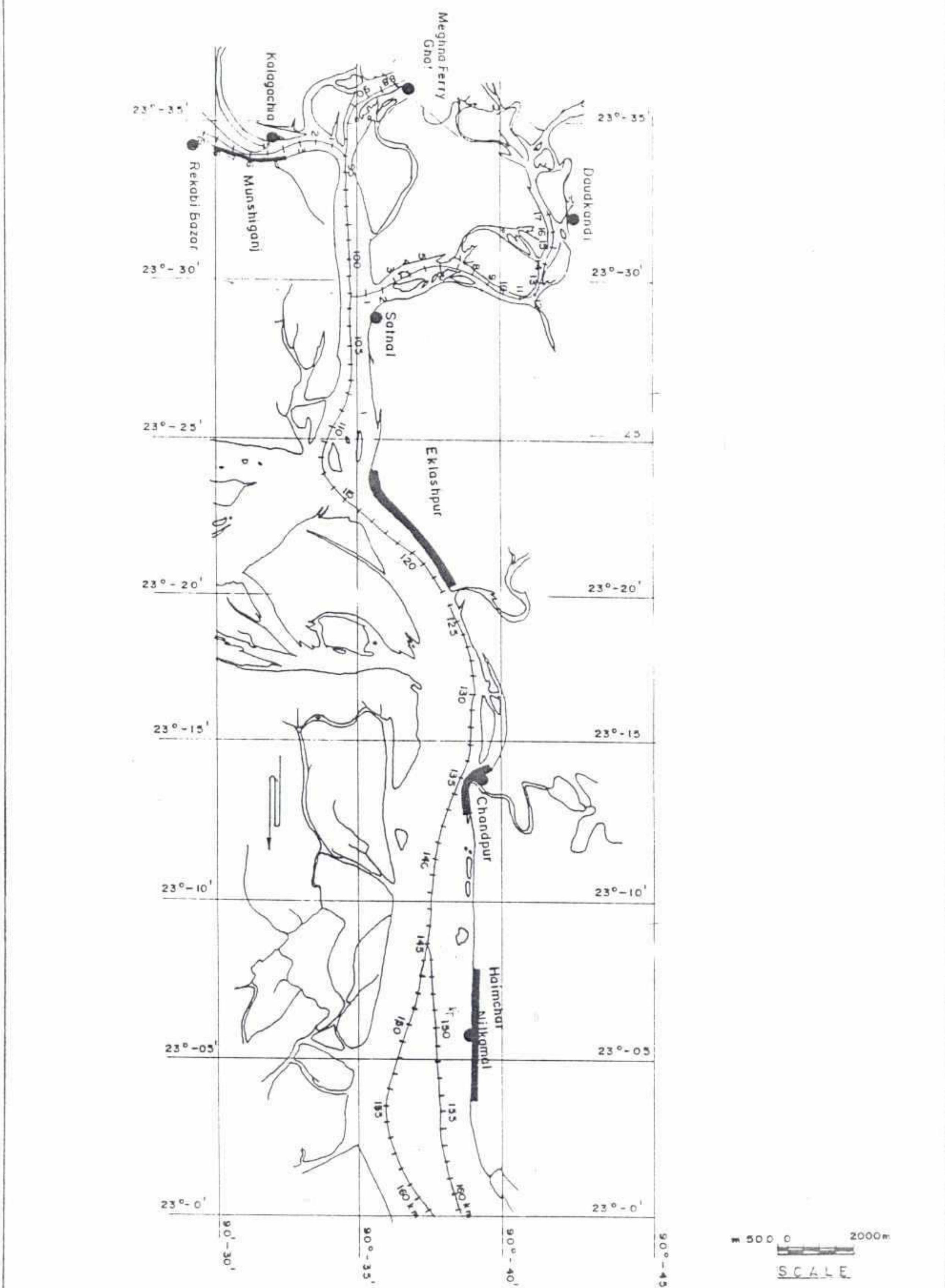
G.2.3.5 Availability of construction materials

For information on the availability of construction materials in Bangladesh reference is made to Chapter G.1, Section G.1.3.4.

G.2.3.6 Topographic and hydrographic surveys

During the month of May 1991 detailed topographic surveys of the banks along the Meghna River at Chandpur were carried out. Typical cross sections of the Meghna River have been surveyed for both Eklashpur and Haimchar in the months of February and March 1991. In November 1991 and December 1991 topographic surveys have been carried out by Consultants at Eklashpur and Haimchar.

Designs have been based on these surveys.



LOCATIONS WHERE BEDSAMPLES ARE TAKEN | FIGURE G.2.3.1

G.2.4 DESIGN CONSIDERATIONS

G.2.4.1 General

(a) Introduction

All river guiding structures contemplated for protecting various interests and/ or infrastructure along the Lower Meghna, as part of this study, have two aspects in common:

- potential scour depths are large, partly being the result of the interaction between river flow and the structure itself (local scour);
- maximum current velocities will be high, whereby the velocities near structures along the Lower Meghna can roughly be twice as high as those occurring on the Upper Meghna (up to 4m/s vs. 2m/s)

In view of the very substantial water depths, structures other than earth structures with reinforced slopes (a protective revetment) will either be unsuitable or be far too expensive (if they can be made at all).

Structures could be made for diverting the river flow (away from a threatened river bank or flood embankment) or for "head on" defence of an attacked (or to be attacked) embankment. The exact location of river defence works depends on the configuration of interests to be defended and the time which will elapse before the attack, if not yet imminent, is expected.

Basically three different situations can be distinguished:

- The structure has to be built in deep water;
  In this case one has not only to take into account the river bed level at the time of construction, but also final scour depths.
- The structure has to be built on land, which is sufficiently far away from the river channel at the time of construction.
- The structure has to be built on the edge of an eroding river bank, which is not situated near deep water, or in relatively shallow water.

In each of the above situations two types of structures could be used to defend embankments or to guide the river flow:

- a groyne type of structure, in most cases to be built perpendicular to the current direction,
- a guide bund, built essentially parallel with the current.

(b) Structures in deep water

Solutions for Chandpur will in any case consist of structures in deep water. The erosive action of the river has resulted in deep scour accompanied by oversteep slopes. Very often the eroded slopes are at the brink of collapse, as has been discussed elsewhere.

A new, stable, slope has to be formed, which can be done by trimming the existing slope (cutting or cut & fill), or by placing a new soil body in front of the existing embankment/ slope.

Construction poses certain problems, such as high current velocities or scour during construction or a short construction window. This situation has also some benefits: scour in excess of the scour present at the beginning of construction is often limited (though certainly not negligible), so that provisions for dealing with the anticipated future scour will not be excessive.

Depending on the flow velocities in the dry season, it may be necessary to construct a temporary protective bund at the river side of the guiding structure to be built. Such a protective bund may, by itself, be a substantial earth structure[5]. For Chandpur such a structure is m[illegible]ely not required in view of the periods with low velocities which will occur during the [illegible] window.

When adopting the principle of implementing an overall bank protection in front of Chandpur problems with acquisition of land for the construction of a proper berm at the top of the new revetment and adequate working space should not be underestimated. Therefore it would be advantageous to consider an advanced protection in order to arrive at a 'smoothly' aligned revetment. For the various alternative solutions proposed reference is made to section G.2.2.3.

A stable slope could be constructed by reclaiming the area by filling and dredging in front of the existing protection works and finish the slopes as steeply as possible. Yet it should be expected that in that case under water slopes may not be steeper than 1:10. Alternatively, it is also possible to place containment bunds with a relatively small bench height under water, and fill this bench with dredged sand.

Considering the water velocities, up to 1 m/s during the construction season, Consultants are of opinion that the first alternative will face difficulties in respect of achieving under water slopes which are steeper than 1:10 to 1:15. Therefore Consultants adopted the concept of the containment bunds.

The concept of the advanced protection is shown in Figure G.2.2.7. Bearing in mind the circumstances when placing the containment bunds a height of each of the bunds of 3.0 m and a slope of 1:1.5 for the landward slope of the bunds has been assumed for estimating purposes (This should however not be imposed on the future contractor who should be free to select the height of the containment bund which best suits his construction method).

Though the containment bunds would consist of coarse material, this material has to be rather finely graded, in order to contain the hydraulic fill. The material of the containment bund will be inadequate to resist the high current velocities during the high water season. After completion of the bunds and the hydraulic fill an appropriate protection is therefore to be placed on the outside of the containment bunds.

G.2.4.2 Eklashpur

G.2.4.2.1 General

A groyne, guiding the flow in the Meghna, which deflects the currents of the Padma (away from the existing embankment of the Meghna Dhonagoda project), appears to be an attractive solution. In view of the very substantial potential scour depths such a groyne should be built in deep water, probably requiring similar techniques as proposed for Chandpur.

Protection of existing embankments, on the other hand, will have to be constructed "on land". The cross section will resemble that of the left guide bund designed for the Jamuna Project, which consist of a trench that has to be excavated. In the trench a proper slope protection can be laid. Moreover, at the toe of the slope protection, at the bottom of the excavated trench, a falling apron is required. This alternative would be constructed in three phases, viz. 1993, 2002 and 2005. The investment in 1993 also includes the protection of the embankment near the mouth of the Dhonagoda River. Construction years take into account the erosion rate as presented in the Geo-morph[illegible] study, Annex B.

A retired embankment with a slope protection, having the shape of a guide bund, will guide the flow of the Meghna River in a more or less similar way as the aforementioned groyne.

G.2.4.2.2 Scour

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5 Such a temporary bund is proposed for one of the guide bunds for the Jamuna Bridge, which is foreseen to be built in a shallow channel of the river.[11]

From the physical model tests. Annex D. it followed that in front of the protection the maximum scour level is 27 (m -PWD). The maximum scour depths referred to the local initial bed level are 10 m. The latter holds for the alternative which has been selected.

G.2.4.2.3 Flow velocities

For Eklashpur model tests show that for the alternative 'protection of the existing banks', maximum flow velocities of 1.9 m/s will occur.

G.2.4.2.4 Wave height

(a) Waves generated by wind

For wave attack a dominant wind direction of NE has been considered with a wind velocity of 29 m/s and a duration of 15 minutes. For the fetch length it is considered that maximum wind velocities will occur in April till June, hence the fetch length will in that case be 7,000 m. For the waterdepth an average value of 30 m is selected. With Bretschneiders formula for wave forecasting a significant wave height of 1.25 m has been calculated. The period applied is 4.18 sec.

(b) Waves generated by ships

Ship traffic on the Lower Meghna will induce wave action, probably not higher than 0.50m. These waves, in view of the ship traffic that can be expected here, are of minor importance if compared to the wind induced waves.

Therefore, for design purposes only wind waves have to be considered, at the condition that these waves are assumed to be present not only during high, but also during low waterlevels.

G.2.4.3 Chandpur

G.2.4.3.1 Dimensioning of structures for Chandpur

(a) Height of bank protection works on existing banks

As already mentioned before, the top of the bank protection will not be beyond the existing shore level as for the design only bank protection works will be considered and no flood embankment. Bank elevations vary from 3.80 (m +PWD) upto 5.70 (m +PWD).

(b) Top of the submerged groyne upstream of Chandpur

In Section G.2.2.3.2(d) the concept of the submerged groyne was introduced. The level of the submerged groyne is 3.00 (m -PWD) whereas the level of the closure dam is higher than the 1:100 years flood level, viz 5.70m. The length of the groyne 2 km upstream of Chandpur would be 600 m and the length of the groyne 3 km upstream of Chandpur would be 800 m.

(c) Groynes in front of Chandpur

The crest of the groynes has been determined considering that it should be higher than the 1:100 years waterlevel, thus resulting in a highest level that should be beyond 5.37 (m +PWD). For the top of the groynes a level of 5.40 (m +PWD) has been selected.

(d) Advanced protection

The width of the berm is approximately 25 m, such providing sufficient working space for the equipment during construction. By applying such a berm unnecessary land acquisition is prevented. The top of the bank should be beyond the 1:100 years waterlevel which implies a level of 5.40 (m +PWD).

G.2.4.3.2 Measures against outflanking and erosion

A process of outflanking will start upstream of the advanced protection at Nutan Bazar. However, based on studies made for the Jamuna Bridge River Training Works, Consultants expect that problems or serious processes endangering the stability will not occur, say within a period of 10 years. Therefore, for the sustainability of the now proposed advanced permanent protection works, it is only necessary to take some measures beyond this period.

According to the Geo-morphological Study, Annex B, near Chandpur erosion will continue if no further bank protection measures are taken. The estimated erosion of the unprotected bank just upstream of Chandpur will proceed at a rate of 30m/year. This erosion will have an intermittent character, as it will be affected by the upstream changes in the channel pattern. Consequently, about 450m in 2004 will be eroded and due to cyclic processes (periodicity of about 15 years) again another 450m will be eroded in 2019. Also for this erosive process measures are necessary in future to provide sustainability of the short-term permanent works.

Section B.8.3.4 of Annex B elaborates on these measures and consequences for the design in more detail.

G.2.4.3.3 Scour

In the Geomorphological Study, Annex B a detailed analysis has been carried out on the geo-morphological processes of the Lower Meghna and its effects on the expected scour depths in front of Chandpur. Also the results of the physical model tests for Chandpur have been used for this study. Since the advanced protection is located in front of Nutan Bazar only aspects related to this location will be dealt with here.

At Nutan Bazar two possible extreme conditions can be identified (see Geomorphological Study, Annex B), notably an upstream bend, and an upstream confluence. For each of these cases an assessment of total scour is made. The probability of occurrence of an upstream bend and an upstream confluence is some 20% and some 10% respectively.

For the possible upstream bend and confluence the contribution of different types of scour has to be assessed. General scour is considered not to be important and constriction scour does not occur at Nutan Bazar. The same holds for bed form scour and local scour. As shown in the Geomorphological Study maximum scour depths will be caused by outer bend scour and protrusion scour. Since these types of scour have also the largest probability of occurrence, they have been used to determine the expected scour depth.

The regime depth at Chandpur is about h_r = 13 m at bankfull discharge and about 1.50m more during 1:100 years flood.

(i) Bend scour

Chandpur is located at an outer bend of the Lower Meghna. The maximum depth resulting from scour at a natural outer bend can be expressed as a function of the average depth of the river. The outer bend scour can then be calculated with:

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$$h_s = k_1 \, h_{av}$$

where:

h_s = *outer bend scour below water level (m)*
k_1 = *factor determined from studies, see Geomorphological Study, Annex B (-)*
k_1 = 1.7 *(according to Geomorphological study, Annex B)*
h_{av} = *average depth (m)*

The average waterdepth h_{av} for Chandpur for a particular discharge in excess of bankfull discharge can be calculated by adding the average depth of the river for bankfull discharge, and the stage difference (= 1.50m) between actual and bankfull discharge:

$$h_{bankfull} = 13.0 \; m$$
$$h_{av\,(1:100\ years)} = 13 + 1.50 = 14.50 \; m$$

The maximum scour to be expected in an outer bend during a 1:100 year flood is:

$$h_s = k_1 \, h_{av}$$
$$h_s = 1.7 * 14.50 = 24.6 \; m$$

The protrusion scour has to be added to this figure.

(ii) Protrusion scour

Protrusion scour can be calculated with the following formula:

$$h_{ps} = k_2 * h_{init}$$

where

h_{init} = *scour due to outer bend scour (m)*
k_2 = 2.8 *(factor see Geo-morphological Study, Annex B)*
h_{ps} = *protusion scour (m)*

In the initial depth the influence of outer bend scour should be taken into account.

By combining both (a) outer bend scour and (b) protrusion scour the maximum local scour, during a 1:100 year flood, can now be determined as follows:

$$h_{init} = (\ 1.7\ (\ h_r + 1.50\ )\ )$$
$$h_{init} = (\ 1.7 * 14.5\ )\ m$$
$$h_s \quad = k2 * h_{init} = 69 \; m$$

This scour depth is referenced to the water level of 5.37 (m +PWD), thus the scour depth below PWD to be expected is about 63 (m -PWD).

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G.2.4.3.4 Flow velocities

Flow velocities for the existing situation, which have been used for design purposes, are derived from Hydrology Study, Annex A. For the determination of flow velocities for various alternative solutions of Eklashpur and Chandpur use has been made of the velocities which have been measured in the physical model tests. In the following Table some characteristic values are presented for Chandpur for design conditions (Q = 130.000 m^3/s).

Table G.2.4.1 RESULTS OF PHYSICAL MODEL TEST CHANDPUR

Alternative solution	Maximum flow velocity (m/s)	Location
Upstream groyne with a length of 1.600 m	2.60	Nutan Bazar
	2.10	Puran Bazar
Advanced protection in front of Nutan Bazar	3.50	Nutan Bazar
	1.50	Puran Bazar

As mentioned in the report on physical model tests, Annex D, there is a significant decrease in flow velocities in front of Puran Bazar, due to the protection in front of Nutan Bazar which deverts the flow away from Puran Bazar.

G.2.3.4.5 Waves

Reference is made for this issue to the discussion on Eklashpur (Section G.2.4.2.4).

G.2.4.4 Haimchar

G.2.4.4.1 General

Haimchar has experienced bank erosion which is probably mainly due to the cutting off process of the large bend that was present in the Lower Meghna River downstream of Chandpur. As is shown in the Geo-morphological Study, Annex B, it seems that this cutting off process is now approaching its final stage. This implies that the erosion rate at Haimchar is expected to slow down. For the time being the erosion rate at Haimchar is assumed to be some 20m/year over the coming decades. For the far future it is extremely difficult to make an estimate without having studied the lower reach of the Lower Meghna River in much more detail. For the time being it is assumed that this slow rate of erosion will continue.

In view of to the future geo-morphological development basically two schemes for protection of the existing embankment are realistic: Alternatives 1 and 2.

The overall length of protection by means of a partly retired embankment, Alternative 1, shaped like a guide bund, is much shorter than the alternative of protecting the existing embankment. Nevertheless Alternative 1 includes extra investment of replacing the embankment.

Both alternatives will in structural and geometrical design resemble the solution for the proposed left guide bund designed for the Jamuna Bridge.

G.2.4.4.2 Scour

For the prediction of scour development in front of the now proposed protection works use is made of the results of the Geo-morphological study and the survey carried out by Consultants in August and September 1991. In Section G.2.5.4.3 the scour at Haimchar is elaborated upon.

x.8

G.2.4.4.3 Flow velocities

As already mentioned before maximum flo.. velocities in front of Haimchar are 1.9 m/s (see Section G.2.3.2.2).

G.2.4.4.4 Waves

Reference is made to this issue as presented for Eklashpur.

G.2.4.5 Soil design parameters

For information on soil design parameters reference is made to Annex C, Geotechnical Investigations.

G.2.4.6 Environmental aspects

During any development of a site from the its initial condition into something different a possibility exists that the environment will be harmed. The Consultants are of the opinion that such a risk should be eliminated in so far as is practically possible. When the risk is above generally accepted standards then the development must be reconsidered or different solutions for the problems examined.

In the Environmental Impact Assessment, Annex I these matters have been elaborated in detail.

G.2.5. **DESIGN**

G.2.5.1 General

G.2.5.2 Geotechnical stability

In Geotechnical investigations, Annex C detailed information is presented on the various geotechnical stability aspects.

The results of extensive slope stability analyses are summarized in Table G.2.5.1

Table G.2.5.1 SAFETY VALUES 1:3.5 SLOPE

Site	slope 1 : 3.5	
	n < 1.5	n < 1
Chandpur	1.68 1.76	1.22 1.27

A slope angle of 1:3.5 does match the design criteria. This slope angle does also match the design requirements when evaluating micro-stability. The stability of an infinite slope will be governed by an internal friction angle $\phi' = 27°$.

G.2.5.3 Slope protection

G.2.5.3.1 Types of slope protection

For the protection of soil structures it is possible to apply "open" or "closed" revetments. In view of the differences in water levels, ground water levels behind the protection and low river water levels, a "closed" protection is not preferred. Therefore only open type structures have been considered.

In Chapter G.1 of this Annex various possibilities of slope protection methods have been evaluated.

G.2.5.3.2 Selection of type of protection

The five criteria which will be considered for selection of type of structure are: (i) functional requirements, (ii) quality assurance, (iii) maintenance, and (iv) construction. For the final selection also a fifth criterion, cost, will be considered.

The criteria are weighed as follows (see Chapter G.1) for obtaining weighing factors:

0.4*i + 0.2*ii + 0.2*iii + 0.2*iv

Two sections are considered:

- protection above water;
- protection under water.

In the following sections both will be discussed in more detail.

(a) Protection above water

Similar to protection works for the Upper Meghna, the following alternatives for protection will be considered for the upper part of the protection above water.

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I Boulders,
II Rock,
III CC-blocks,
IV Block mattresses,
V Open stone asphalt on geotextile.

In the following Table the scores 1 to 5 indicate the relative suitability of the alternative, whereas 5 indicates 'fully satisfies requirements' and 1 indicates 'insufficiently at all times'.

Table G.2.5.2 MULTI CRITERIA ANALYSIS UPPER PART OF THE PROTECTION

		I Boulders	II Rock	III CC-blocks	IV Block mattresses	V Open stone asphalt
i	Functional requirements	2	2	3	4	5
ii	Quality assurance	4	3	4	1	2
iii	Maintenance	2	2	2	4	5
iv	Construction	4	4	3	1	2
TOTAL		7.20	5.60	7.32	1.64	9.20
Ranking		3	4	2	5	1

In the following criteria for selection are given.

(i) Functional requirements

Open stone asphalt and block mattress are most suitable to withstand the waves. CC-blocks have also a good resistance to waves. Next in order are boulders followed by rock. The latter is not very effective compared to the other types of revetment mentioned.

(ii) Quality assurance

For the upper part it will be easy to place simple elements such as boulders and CC-blocks. Block mattresses require more sophisticated methods.

(iii) Maintenance

Maintenance is more or less the same for all types of revetments. However, some differentiation can be made. Block mattresses are favoured because elements are connected and can not be easily taken away. The latter is also true for grouted elements. The order of suitability regarding maintenance is: gabions, CC-blocks, boulders.

(iv) Construction

Construction and/or placing of boulders, gabions and CC-blocks is quite easy. It does not require much working space or skill.

(b) Protection under water

The following alternatives will be considered for the under water protection.

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I Boulders,
II Rock,
III CC-blocks,
IV Block mattresses,
V Grouted elements.

Table G.2.5.3 MULTI CRITERIA ANALYSIS UNDER WATER PROTECTION

	I Boulders	II Rock	III CC-blocks	IV Block mattresses	V Grouted elements
i Functional requirements	3	5	3	2	2
ii Quality assurance	3	2	2	4	4
iii Maintenance	3	4	2	1	5
iv Construction	4	3	3	1	2
Ranking	2	1	4	5	3

In the following criteria for selection are given.

(i) Functional requirements

Boulders and (more or less equivalent) rock are most suitable for the under water protection in relation to their function. Because of their rounded shape, boulders easily start rolling on a slope if compared to rock or CC-blocks. The latter have both a higher degree of interlocking. Grouted elements and block mattresses could also be applied but are not really required.

(ii) Quality assurance

Block mattresses will be easy to construct. The accuracy of placing and monitoring grouted elements is quite high. The differences between boulders, rock and CC-blocks are rather small and are mainly caused by the weight.

(iii) Maintenance

Boulder and rock under water will not require much maintenance. Because of the strength of block mattresses maintenance will also be low. However, connections between the elements can give problems. The maintenance of grouted materials is very low.

(iv) Construction

The construction of a revetment incorporating grouted materials requires special equipment; the same holds for block mattresses. For such construction equipment foreign contractors will be required. For Chandpur conditions the boulders and rock do also require foreign contractors. The other protection types, as they have to be placed with high accuracy require also special equipment.

(c) Results of MCA on selection of revetment

Based on the results of the MCA and considering the cost of the various types of protection Consultants propose to adopt the following:

- for the upper part protection above water: stone asphalt on geotextile.
- for the underwater protection: rock on a fascine mattress.
- for the falling apron section: rock having a proper grading without a underlying geotextile.

Dimensioning of the falling apron will be dealt with in the following Section.

G.2.5.3.3 Resistance against curren' attack

For the designs use will be mad- -' ...e formulae which have been recently developed in the Netherlands and adjusted and applied to the Jamuna Bridge Project in Bangladesh.

Use is made of the Pilarczyk formula for the stability of cover layers under current attack [1].

$$\Delta_m D_n = \phi K_t \frac{0.035}{\psi_{cr}} \frac{K_h}{K_s} \frac{\overline{u}^2}{2g}$$

where:

Δ_m = *relative density* (-)
D_n = *dimensions of cover elements* (*m*)
ϕ = *stability factor* (-)
K_h = *depth factor* (-)
K_h = $(h/D_n)^{-0.2}$
k_r = D_n (*smooth units*) (*m*)
k_r = $2\ D_n$ (*rough units*) (*m*)
K_s = *slope factor* (-)
K_s = $((1-\sin^2(\alpha)/\sin^2)^{0.5}(\theta)$ (-)
$\overline{u}$ = *mean ve'ocity* ϵ *vertical near bank*(*m/s*)
g = *accelaration of gravity* (m/s^2)
K_t = *turbulence factor* (-)
ψ_{cr} = *critical shear stress* (-)
θ = *angle of internal friction* (*degrees*)
α = *angle of slope protection* (*degrees*)

The weight of the boulders can be derived from the dimensions of a stone according to:

$$D_{50} = 1.18\ D_n$$

$$M_{50} = D_n^3\ \rho_s$$

where

M_{50} = *mass of which is being exceeded by* 50 % *of the total mass of the batch of stones* (*kg*)
D_n = *nominal stone diameter* (*m*)
D_{50} = 50% *value of size distribution* (*m*)

Firstly a deterministic design method will be applied. In Section G.2.5.6 probabilistic calculations will be performed to improve the designs.

In the following Table each of the parameters involved in the determination of the boulders size, will be discussed for the bank protection works at Chandpur Town.

Table G.2.5.4 VALUES TO USE FOR FORMULA (CURRENTS)

Parameter	Value	Remarks
$\bar{u}$	3.50 m/s	According to the results of phys.cal model test the 1:100 years design velocity is 3.50 m/s for Chandpur Town area.
Δ_m	1.65	For Chandpur the density of the water will be 1,000 kg/m^3 whereas per specification of the BWDB the specific density of the rock is 2650 kg/m^3. For the calculation of the relative density use has been made of these figures.
α	1:3.5	For geotechnical stability a slope of 1:3.5 is recommended. All slopes should stay at or below this value.
h	40m	An average waterdepth of 40m has been adopted.
K_t	1.8	Consultants expect fairly high but not excessive turbulence in this area up to 20%. They recommend K_t = 1.8 for the [illegible]istic calculations. Probabilistic calculations will be performed with an average value of 1.5 and a standard deviation of 0.15
θ	40°	According to Lanes graph an angle of 40° has been used in the Pilarczyk formula for rock.
Φ	1.00	Application of rock results in a stability factor of 1.00 due to irregularities at connections of mattresses.
ψ_{cr}	0.035	The critical Shields shear stress parameter which has been applied is 0.035.

When substituting these values in the formula rock with a characteristic diameter of 0.32m is found. Rock with D_{50}=0.35m has been selected with a layer thickness of 0.70m. Underneath the rocks a geotextile is placed onto which a grid of fascines (wood, reed, bamboo) may be attached to facilitate settling in of the loose material (see Figure G.2.5.1). The final dimensioning will be done after the probabilistic calculations have been performed.

G.2.5.3.4 Resistance against wave attack

For the upper part of the protection, for dimensioning the revetment to withstand wave attack use is made of following formula which includes the effect of the wave period [1].

$$\frac{H_s}{\Delta_m D} = \psi_u \frac{c}{\sqrt{\xi_z}}$$

where:

ψ_u - *upgrading factor* (-)
ϕ - *stability factor* (-)
H_s - *significant wave height* (*m*)
ξ_z - *wave parameter* (-)

$$\xi_z = \frac{\tan \alpha}{\sqrt{H_s/L_0}} = \frac{1.25\ T_z \tan\alpha}{\sqrt{H_s}}$$

L_0 - *wave lenght* (*m/s*)
T_z - *wave period* (sec)
α - *slope*($\angle$)
Δ_m - *relative density* (-)
D - *thickness of protection* (*m*)

The thickness of the open stone asphalt layer can be derived by filling in the values as given in Table G.2.5.5.

Table G.2.5.5 VALUES TO BE USED FOR FORMULA FOR OPEN STONE ASPHALT(WAVES)

Parameter	Value	Remarks
H_s	1.25m	For wave attack a dominant wind direction of NE has been considered with a wind velocity of 29 m/s. For the fetch length it is considered that maximum wind velocities will occur from April to June, hence the fetch length will in that case be 7,000 m. For the waterdepth an average value of 30 m is selected. With Bretschneiders formula for wave forecasting a significant wave height of 1.25m has been calculated.
T_s	4.18 sec	With Bretschneiders formula for wave forecasting a period of 5.5 sec has been calculated
ξ_z	1.00	This parameter is set at 1.0 for open stone asphalt
Δ	1.00	This parameter is set at 1.0 for open stone asphalt.
α	1:3.5	For geotechnical stability a slope of 1:3.5 is required.
ψ_u	6.00	This parameter is set at 6.0 for open stone asphalt..
ϕ	1.00	This parameter is set at 1.0 for open stone asphalt.

When substituting these values in the formula a minimum value for the thickness of the layer is found of 0.20 m.

The open stone asphalt layer extends to 2.50 (m +PWD) and allows placing under dry conditions. This level corresponds approximately with a waterlevel that is exceeded 50% of the time (see Hydrological Study, Annex A).

The lining consists of a layer of 0.20 m thick on a filter layer. This filter layer consists of a synthetic woven filter fabric (see Figure G.2.5.2).

G.2.5.3.5 Counter measures against scour

For detailed information on the application of the falling apron reference is made to Chapter G.1. of this Annex. In the following the dimensions of the falling apron are listed for Chandpur.

The dimensions of the rock have been determined by applying the same formula as for the attack of the upper part. By replacing the slope of 1:3.5 and the internal stability in the aforementioned calculation by 1:2 and 40 respectively a diameter of the rock of 0.33m is found. Furthermore Kt has been decreased to 0.75 because the falling apron is a continuous protection.

Table G.2.5.6 DIMENSIONS OF FALLING APRON SECTION CHANDPUR

Ds (m)	Lf (m)	Qf (m^3/m)
14.50	23.50	82.00

G.2.5.3.6 Filter requirements

(a) Granular filters

A granular filter between subsoil and outer layer has to meet the following requirements, related to the representative grain sizes of the subsoil Db and the filter Df:

Table G.2.5.7 GRANULAR FILTER CRITERIA

Criterium	Constraints
permeability	D15f/D15b > 4 - 5
segregation	D50f/D50b < 20 - 50
piping	D15f/D85b < 4 - 5
internal stability	D60f/D10f < 10 no migration D60f/D10f > migration

For the time being no granular filters are considered.

(b) Geotextiles

Geotextiles are more and more used as separation between layers of different composition. Both woven and non-woven geotextiles can be considered. In the following Table the requirements which are used for the review are summarized:

Table G.2.5.8 CRITERIA FOR GEOTEXTILE FILTERS

Type of geotextile	Sandtightness	Permeability
Woven	O90/D90 < 1	kf = 5 ksoil
Non-woven	O90/D90 < 1.8	kf = 5 ksoil

According to the soil data valid for Chandpur the governing characteristic soil parameter is D90 = 0.09*10-3 m and the permeability parameter will be ksoil = 6.0*10-7 m/s.

For the sand-tightness and permeability of the woven geotextiles the following criteria are adopted [3]:

$$O_{90} < O_{90b}$$

where:

O_{90} - *effective opening size of geotextile (m)*
O_{90b} - *characteristic size of subsoil particles (m)*

$$k_t = 5\ k_{soil}$$

where

k_t - *permeability geotextile (m/s)*
k_{soil} = *permeability soil (m/s)*

To meet the requirements a composite geotextile is required which consists of a combination of a woven, for the strength, and a non-woven, for the sand tightness.

Soil characteristics applied here are D_{90}=0.09*10-3 m and k=6.0*10-7 m/s (see Section G.2.3.3.). In the following Table specifications for woven geotextiles are listed.

Table G.2.5.9 GEOTEXTILE SPECIFICATIONS WOVEN (TYPE I)

Item	Specification
Type of geotextile	100 % woven polypropylene
Effective pore size	200*10-3 < O90 < 300*10-3 m
Permittivity	> 0.1 s-1
Strength wrap and weft	70 kN/m
Weight	450 gr/m2

In the following Table specifications for non-woven geotextiles are listed.

Table G.2.5.10 GEOTEXTILE SPECIFICATIONS NON-WOVEN (TYPE II)

Item	Specification
Type of geotextile	100% non-woven
Effective pore size	O90 < 0.125 10-3 m O50 < 0.075 10-3 m
Permittivity	> .1 s-1
Strength	> 70 kN/m
Weight	> 200 gr/m2
Grab strength	> 900 N
With on roll	> 5 m

This composite geotextile will be placed under the open stone asphalt layer and integrated in the fascine mattress.

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G.2.5.4 Design of various alternative schemes at these locations

G.2.5.4.1 Eklashpur

(a) Evaluation

For the various Eklashpur alternatives design drawings have been made. In one of the previous Sections the groyne alternative (see Figure G.2.5.3) has been presented. A design was also made for the retired embankment shaped like a guide bund (see Figure G.2.5.4), and the protection of the existing embankment.

Finally, the protection of the overall bank has been considered but it was decided not to carry out a further analysis because of its low protective effect in relation to its enormous length.

For the aforementioned alternatives cost estimates have been made. Construction has been phased so investments do not need to have made in one year (see Chapter G.2.7). As it was felt that Eklashpur is most benefitted by a solution that can be implemented relatively quick and of course is economically feasible, Consultants selected the alternative of the protection of the existing embankment for further elaboration.

The first phase consists of the protection of the existing embankment at Eklashpur (length=1,300m) and the protection of the existing embankment in the mouth of the Dhonagoda River (length=900m). In the following only the first phase of construction is dealt with. The first phase will comprise short term protection works which can be constructed at short notice and which will be permanent structures. Sustainability ultimately should be guaranteed by the implementation of protection works of the said second and third phase.

(b) Design and dimensioning

Since the maximum current velocities (1.9 m/s according to physical model tests) and the maximum scour depths to be expected (10 m also according to physical model tests) at Eklashpur are similar to those at Bhairab Bazar, the same type and dimensions of slope protection under water and of falling apron are applied, i.e. in both cases D_{50} = 0.15 m. The underlying mattress and filters are also the same. Because wind attack of a higher magnitude is to be expected than at Bhairab Bazar (see Annex A) the thickness of the open stone asphalt is 0.20 m.

The open stone asphalt layer extends to 2.50 (m +PWD) and allows placing under dry conditions. This level corresponds approximately with a waterlevel that is exceeded 50% of the time (see Hydrological Study, Annex A).

As mentioned before this protection will be partly build in the dry by excavating a trench. The bottom level of this trench is 7 (m -PWD). Selection of this level has been based on the topographic survey carried out by Consultants in November 1991.

In Figure G.2.5.5 the layout of the proposed protection works is presented whereas in Figure G.2.5.6 a typical cross section is presented. The protection of the BWDB protection has been extended in the lower part by a boulder mattress and a falling apron section, as shown in Figure G.2.5.7.

G.2.5.4.2 Chandpur

(a) Selection

The groyne alternative upstream of Chandpur Nutan Bazar is shown in Figure G.2.2.11 whereas a layout plan of the advanced protection is presented in Figure G.2.2.8.

The layout for the advanced protection in front of Nutan Bazar is shown in Figure G.2.5.8. A typical cross section is shown in Figure G.2.5.9.

Some of the alternatives can be considered as part of long term solutions having a 'regional' effect. The other solutions are of local nature, do not require additional comprehensive further studies and can be implemented in the near future. As it was felt that Chandpur is most benefitted by a relatively quick short-term permanent solution the advanced protection of Nutan Bazar was selected for further elaboration. Selection of this solution also requires protection works at Puran Bazar.

(b) Design and dimensioning

(i) Chandpur, Nutan Bazar

The dimensioning of the advanced protection in front of Chandpur Nutan Bazar has been discussed in the previous sections.

Additional works at Puran Bazar will be presented in the following. Finally, measures will be presented to ascertain the sustainability of the now proposed advanced protection in front of Chandpur Town.

(ii) Chandpur, Puran Bazar

For Puran Bazar river bank protection works are required in addition to those at Chandpur, Nutan Bazar. On the basis of bank profiles it is concluded that the approach must be similar to that applied for Chandpur, Nutan Bazar. Due to the shape and alignment of the proposed advanced protection at Nutan Bazar velocities in front of Puran Bazar will be low compared to those at Nutan Bazar and the present situation at Puran Bazar. The results of the scale model tests are very clear in this respect and show velocities of 1.5 m/s in front of Puran Bazar.

Slopes of 1:3.5 are required to obtain a stable slope. In some locations at Puran Bazar this requires filling with sand of D_{50} > 150 mu. Depending on the steepness of the slope and the depth this can be done either by the concept of the containment bunds and filling or just filling without containment bunds. The latter approach is similar to that applied at Bhairab Bazar.

For current attack boulders of D_{50}=0.15m (Grade I) on a fascine mattress (Geotextile Type I and II) can be applied. For the upper part open stone asphalt with a thickness of 0.20m on a geotextile Type I will be sufficient to withstand wave attack. Between the open stone asphalt and the boulders on the fascine mattress the boulders will be grouted with bitumen. The falling apron section consists of graded boulders of D_{50}=0.15m (Grade II). Dimensions of the falling apron are the same as for Bhairab Bazar.

In the area just South of Puran Bazar, where depths are not more than approximately 5.00m and where slopes are not very steep, only protection against wave attack will be sufficient. The protection proposed consists of the same open stone asphalt and bitumen grouted boulders as mentioned before with a thickness of 0.20m on a geotextile Type I. The width of this open stone asphalt protection will be approximately 30m.

In Figures G.2.5.10 and G.2.5.11 a layout and a typical cross section of the protection works at Chandpur Puran Bazar have been presented.

The banks in the mouth of the Dakatia River will also be protected as part of the now proposed protection works at Nutan Bazar and Puran Bazar. No hindrance to navigation is to be expected as shown in Figure G.2.5.12 where a cross section has been presented of Dakatia River, showing three typical vessels with maximum dimensions (draught = 2.80m and width = 9.68m according to BIWTMAS, November 1988).

In Figure G.2.5.13 the overall layout of the short term protection works for Chandpur are shown.

(iii) Future measures to ascertain sustainability of the short-term permanent works

As mentioned before measures in respect of the erosion and the outflanking should be planned to ascertain the sustainability of the protection works of Chandpur Town in the coming 30 years. On the basis of the information discussed in Section B.5.3 (Annex - B. Geo-morphological Study) it is possible to make a prediction of the future developments of the planform of the Lower Meghna River. These most probable future developments are analyzed in order to take them into account in the design of the short-term permanent protection works along the Lower Meghna. In doing this a distinction is made between the conditions upstream of Chandpur and the conditions in the downstream reach. A detailed evaluation of the geomorphological changes should be included in the TOR for the Long-Term Strategic Plan.

The estimates presented in Annex B can also be used to estimate the bank erosion along the Lower Meghna at Eklashpur. This is done in Section B.6.3.2 in a detailed manner. Here it suffices to conclude that this bank erosion will proceed as a kind of cyclic process with a periodicity of some 15 years. Around the year 2006 erosion of another 700 m (approximately!) will have taken place, and around 2021 another 700 m (or more if the curvature of the channel along Eklashpur decreases).

This erosion will not be limited to Eklashpur but a reach of some 10 to 20 km especially south of Eklashpur will be eroded gradually. It should be stressed here that this will not be a continuous process, that will proceed at a constant speed. Due to the periodic nature, but also due to the additional effect of second-order bars in the system, the bank erosion will be intermittently and may even be marked by short periods of accretion. The overall trend however will be erosion.

As estimated above without bank protection works parts of Chandpur town will gradually be eroded. At present the estimated rate of bank erosion of the area which has been protected heavily in the past is estimated to be 20 m/year. The adjacent areas will also erode although at a lesser rate than along the outer bend near Eklashpur. For the time being a rate of some 30 m/year (on the average) is assumed to be a fair estimate.

In the river reach near Haimchar still some erosion is estimated to occur. This erosion process however, will gradually proceed at a slower rate, as this erosion is due to the further widening of the cutoff channel. This process is almost complete. Still erosion in the order of (on the average!) some 20 m/year is expected over the coming decade. Hence the overall trend of the Lower Meghna River over the coming decades will be a continued movement towards the east, with intermittent periods of reduced erosion or possibly even some accretion from time to time. Therefore, as mentioned before measures in respect of the erosion and the outflanking should be planned to ascertain the sustainability of the short-term permanent protection works of Chandpur Town in the coming 30 years.

An estimate of the extent of the area of outflanking upstream of Chandpur has been made on the basis of the previous studies (scale model) carried out by the Consultants for the guide bund at the Jamuna Bridge. For this purpose experiments were carried out in an overall movable bed model of the Jamuna River. In addition the results obtained from the 2-D model were also examined (flow field, which results from the depth averaged flow computations). The process of outflanking could affect an area limited to 400m having a diameter of approx. 400m in the near future. As the erosion at Chandpur will be approximately 450m in 2004, Consultants propose to construct a bank revetment of 400m in 2003 adjacent to the advanced protection now proposed (see Figure G.2.5.14).

For the time being the revetment can be more or less similar to the one now proposed for Chandpur Nutan Bazar.

G.2.5.4.3 Haimchar

(a) Evaluation

The two basic solutions for Haimchar, protection of the existing embankment by means of a guide bund like structure and protection of the existing embankment, see Section G.2.4.4.1, have been considered. Both the solutions deflect the flow lines in more or less the same way. Therefore also the effect on the erosion process will be comparable and the dimensions of the structures can be more or less the same. Following the economic evaluation (see Annex F) it can be concluded that the solution of part protection of the existing embankment and part replacement is most beneficial for Haimchar. Consultants therefore propose to adopt this alternative.

Similar to Eklashpur also the construction of the protection works at Haimchar are presented in a phased manner (see Section G.2.7.1.3). Only the first phase of construction is discussed here. These short term protection works of the first phase can be implemented at short notice and will be permanent works. Therefore, similar to Eklashpur, protection works of the second and third phase are ultimately required to ascertain the sustainability of the short term measures of the first phase.

(b) Design and dimensioning

Similar to Eklashpur the protection will be constructed in the dry in a excavated trench. The bottom level of this trench is 7.0 (m -PWD). Selection of this level has been based on the topographic survey carried out by Consultants in November 1991.

The maximum scour level to be expected in front of the protection is 13 (m -PWD). Since the design bottom level of the trench is 7 (m -PWD) a scour depth of 6m can be expected for a unchanged situation. According to the Geo-morpholigical Study the erosion process will decrease at Haimchar area. As the aforementioned scour depth of 6m is based on an unchanged situation (as well as the prediction of the future Geo-morphological development) Consultants adopt a scour depth of 10m for the design of the falling apron section. This results in dimensions of the falling apron equal to those of the design at Eklashpur, i.e. 24 m^3 per lin.m.

Other boundary conditions, viz. flow velocities of 1.9 m/s and wave heights of 1.25m, for Haimchar are similar to those at Eklashpur. Therefore Consultants made designs of the slope protection works which are similar to those at Eklashpur.

In Figures G.2.5.15 and G.2.5.16 respectively the layout of the protection works and a typical cross section are presented.

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G.2.5.5 Risk Analysis

G.2.5.5.1 Introduction

For the design of the bank protection a risk analysis has been carried out. The objectives of the Risk Analysis are:

- to define an acceptable probability of failure of the bank protection;
- to identify and quantify the hazards of the bank protection;
- to integrate the design of the bank protection into other infrastructural works.

In Chapter G.1 of this Annex and in Appendix G/5 a more elaborated description is given of the risk analysis.

G.2.5.5.2 Failure modes

An overall fault tree for the advanced bank protection in Chandpur is presented in Figure G.2.5.17. For more information on failure modes reference is made to Section 1.5.4.2 of Chapter G.1. The specific site of Chandpur has been considered when this fault tree was prepared. Outflanking is not a major failure mode in the coming few years and therefore does not figure in the fault tree. As mentioned before after some years this failure mode should be taken into consideration. An elaborate discussion on outflanking has been presented in Section G.2.5.4.2.

G.2.5.5.3 Acceptable probability of failure

In respect of failure probabilities for structures at Eklashpur and Haimchar, when considering the losses (see Figure G.1.5.11), the values hold as presented for Bhairab Bazar, viz. 5.0*10-3. Fault trees for the protection works at Eklashpur and Haimchar are also similar to the one presented for Bhairab Bazar (reference is made to Figure G.1.5.10).

The acceptable probability of failure of a overall bank protection for Chandpur is discussed in the following.

In Chapter G.1, Section G.1.5 a graph frequently used for determination of acceptable risk levels for various structures and activities is shown [1]. Considering the type of protection, magnitude of loads and the commercial areas which are in danger, for Chandpur an acceptable failure probability of 0.25*10-3 has been selected. Note that for the sites considered along the Upper Meghna a failure probability of 0.5*10-3 has been adopted. The difference is due to the higher value of interest in chandpur.

Starting at this value of failure probability of 0.25*10-3 for a bank protection the failure probabilities of the different components of the fault tree can be determined. In the following this iterative exercise of distribution of the failure probabilities is discussed.

As the slope stability calculations indicate that for the selected slopes of 1:3.5 the safety coefficients are sufficiently high, the failure probability is quite low (see Geotechnical Investigations, Annex C). Nevertheless, for safety reasons the probability for geotechnical failure is set at 0.125x*10-3.

The probability of 2.0*10-1 for failure due to lack of monitoring and maintenance is based on sound engineering judgement.

In the following sections this value is maintained as criterion for the probabilistic calculations.

G.2.5.6 Probabilistic calculations Chandpur

G.2.5.6.1 Current attack

For the determination of the characteristic diameter of a stone or concrete cement block on a slope, the formula which has been presented in the previous Section has been applied for the probabilistic calculations.

In the previous Section it was concluded that according to the prevailing criteria probability of failure of an element of a bank protection should not be more than indicated for the respectively failure modes in Figure G.2.5.17. The reliability function which has been applied can be described by:

$$Z = \Delta_m D_{50}\, 0.847 - \phi\, K_t \frac{0.035}{\psi_{cr}} \frac{K_h}{K_s} \frac{\bar{u}^2}{2\,g}$$

The parameters have been defined as presented earlier. The average current velocity in a vertical profile is derived from the Chezy formula:

$$\bar{u} = C\,\sqrt{(h\,I)}$$

where:

I = *slope* (-)
C = *Chezy value*$(m^{1/2}/s)$

To determine the waterdepth use is made of the stage relationship as presented in the Hydrological Study, Annex A. For the discharges use is made of a Gumbel extreme value distribution (A=11,955, B=97,658) for the discharges which occur during floods for Chandpur area. The values for the extreme value distribution have been obtained form the Hydrological Study, Annex A.

The parameters, except these for the discharges, have a normal distribution, characterised by an average and a standard deviation. From these parameters, other parameters can be derived via simple relationships.

For several combinations of slope, average and standard deviation of D_{50} calculations have been performed. In Appendix G/4 some results are shown. In the following Table the final results are summarized.

Table G.2.5.11 RESULTS OF PROBABILISTIC CALCULATIONS RESISTANCE AGAINST CURRENT ATTACK

Section	Slope	$\mu(D_{50})$ (m)	$\sigma(D_{50})$ (m)	Probability of failure (1/year)	Acceptable probability of failure (1/year)
Falling apron	1:2	0.35	0.035	4.21*10-3	1.56*10-2
Lower part	1:3.5	0.35	0.035	2.09*10-5	1.56*10-2
Upper part	1:3.5	0.35	0.035	2.53*10-3	3.13*10-3

The calculated probabilities show that the acceptable failure probabilities for the proposed dimensions are not exceeded. Thus for the rock on the fascine mattress as well as for the falling apron a $D_{50}=0.35$ has been selected.

G.2.5.6.2 Wave attack

It is assumed that the probability density function for the wave heights can be described by a Gumbel extreme value distribution (A=0.721, B=0.110). For the determination of the characteristic diameter of a stone on a slope, the formula for wave attack which has been presented in Section G.2.5.3.4 has been applied for the probabilistic calculations. Slightly rewritten this formula is as follows:

$$Z = D - \frac{H_s}{6.0}$$

The main results of the probabilistic calculations (see Appendix G/4) for the open stone asphalt cover layer are presented in the following Table.

Table G.2.5.12 RESULTS PROBABILISTIC CALCULATIONS WAVES (OPEN STONE ASPHALT)

Slope	$\mu(D_{50})$ (m)	σ(D50) (m)	Probability of failure (1/year)	[6]Acceptable probability of failure (1/year)
1:3.5	0.20	0.020	1.47*10-1	1.50*10-1

As shown in the Table the probability of failure does not exceed the acceptable probability of failure.

G.2.5.6.3 Scour depths

The design of the bank protection shall be based on a combination of various forms of scour. Use has been made of the formula presented in Section G.2.4.3.3.

For the determination of the probability of failure of a falling apron having a certain scour depth the following reliability function has been applied:

6 The value presented is different form the one presented in the fault tree. A commonly used criterion for wave attack at open stone asphalt is 1.5*10-1.

$$Z = H_{tot} - H_{init} - k_3\, q^{0.67} + 16.2 - h_{gen}$$

$$H_{tot} = H_{level} - Scourlevel$$

$$H_{init} = k_1\, h_{av} + k_2\, H_{constr}$$

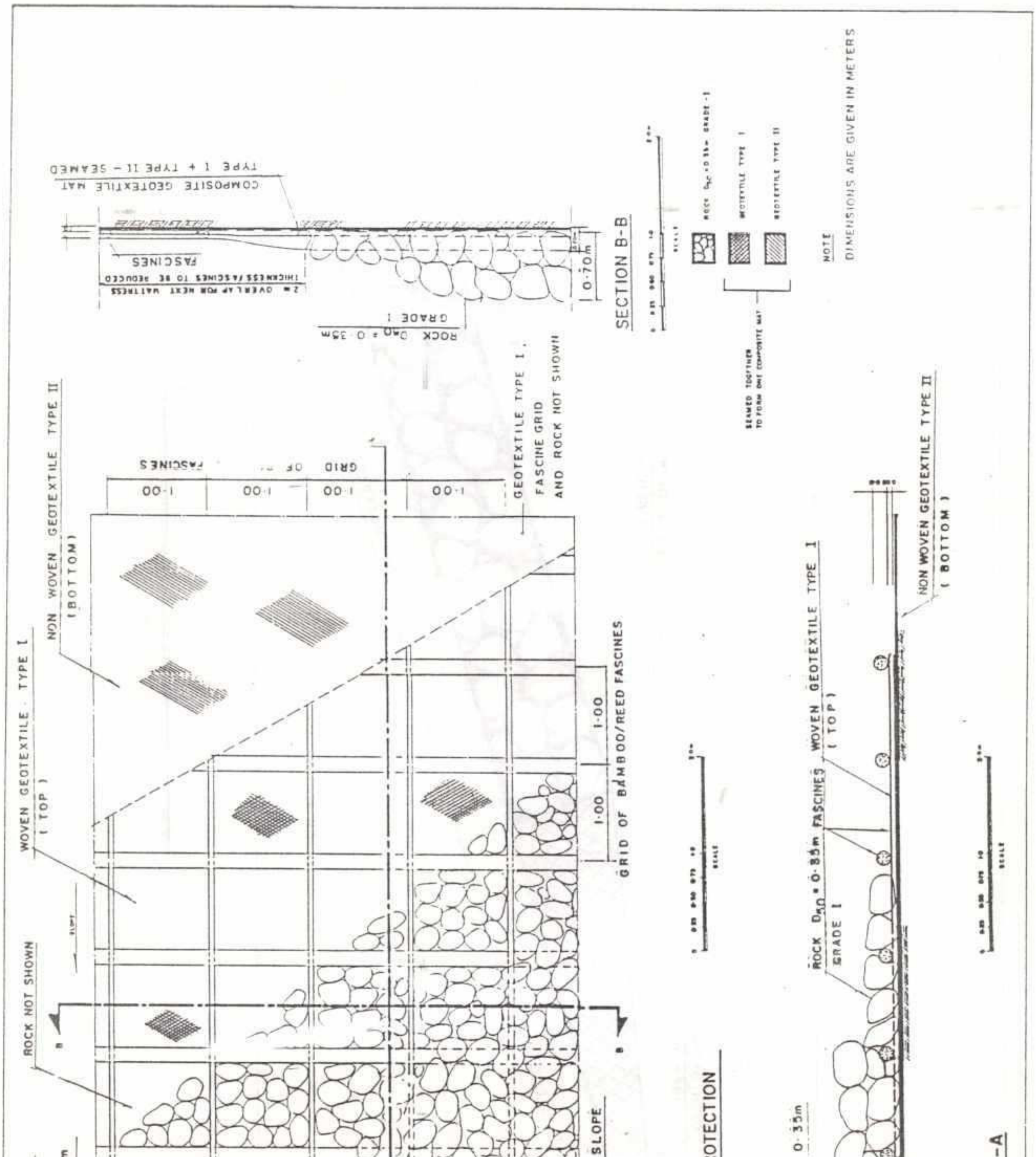
For the discharge use is made of a Gumbel extreme value distribution. The other parameters follow a Gauss distribution.

The following scour depths along the bank protection and corresponding probabilities of failures have been determined. The results are listed below.

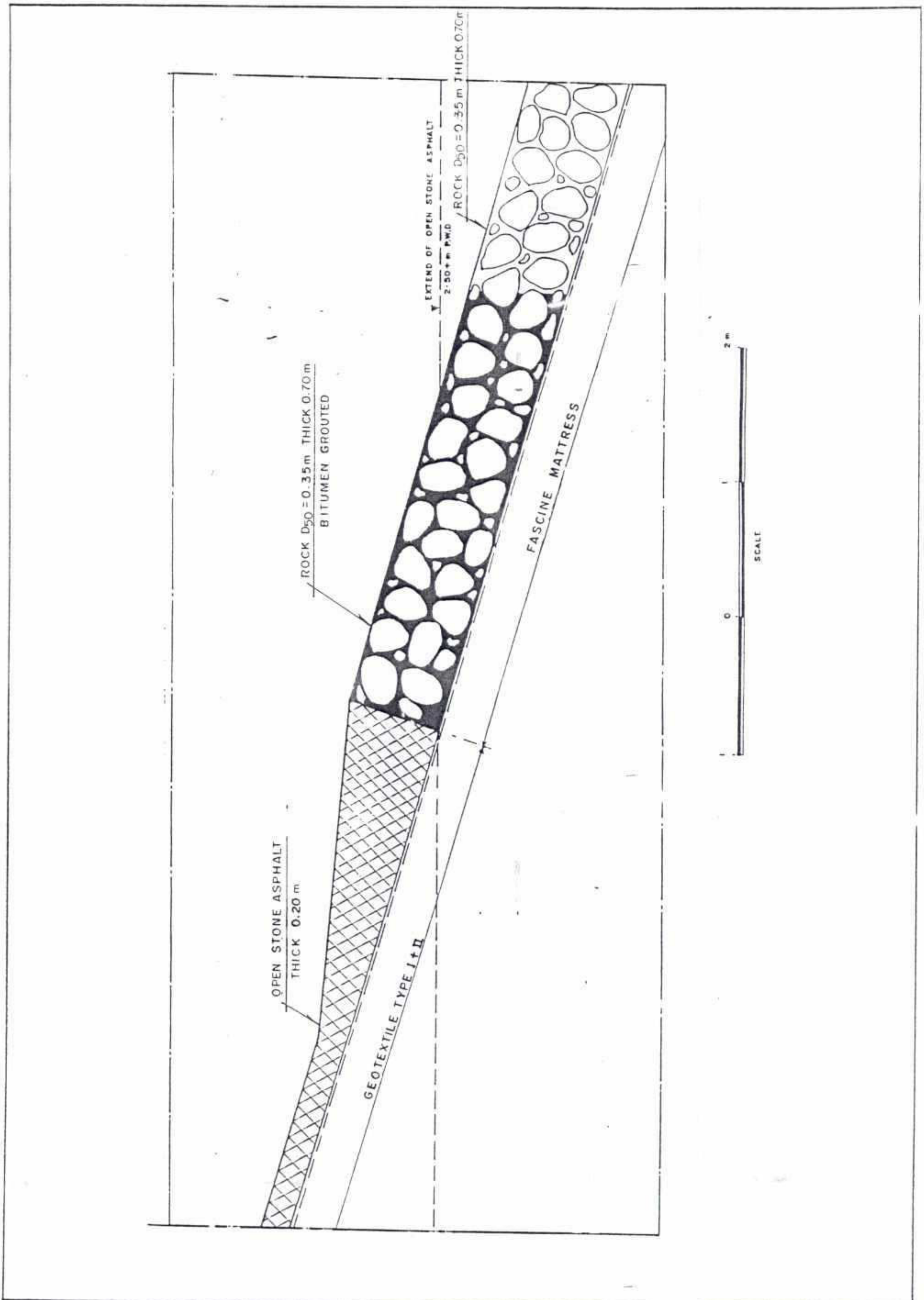
Table G.2.5.13 RESULTS PROBABILISTIC CALCULATIONS EXPECTED SCOUR DEPTHS

Scour level referred to initial bed level (m)	Probability of failure (-)	Acceptable probability of failure (-)
14.50	6.3*10-2	7.00 10-2

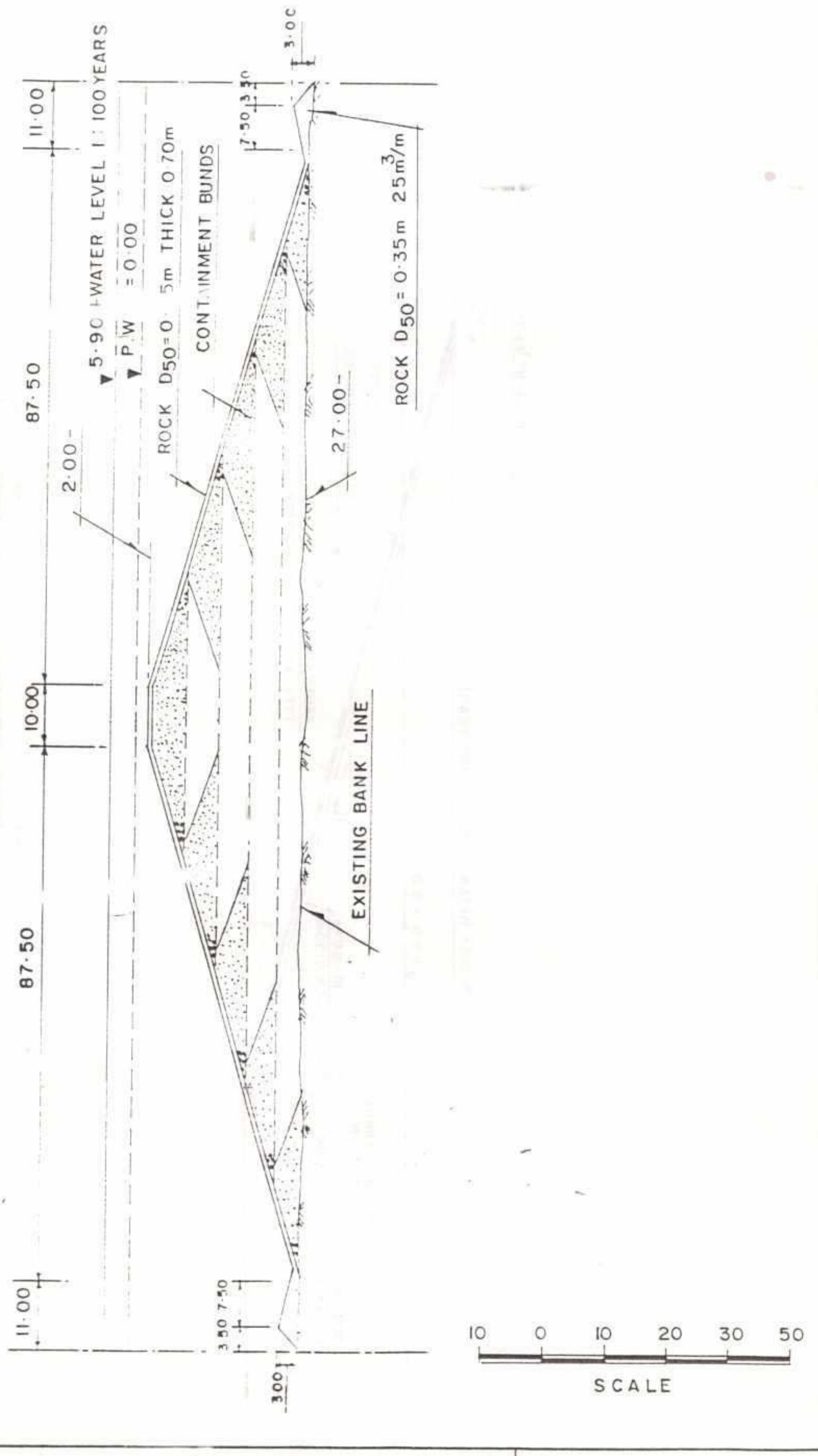
As shown in the Table the probabilities of failure are lower than the acceptable probability of failure. In Appendix G/4 more results of the probabilistic calculations are presented. For the falling apron a design scour depth referring to the initial bed level 0f 14.5m has been applied.



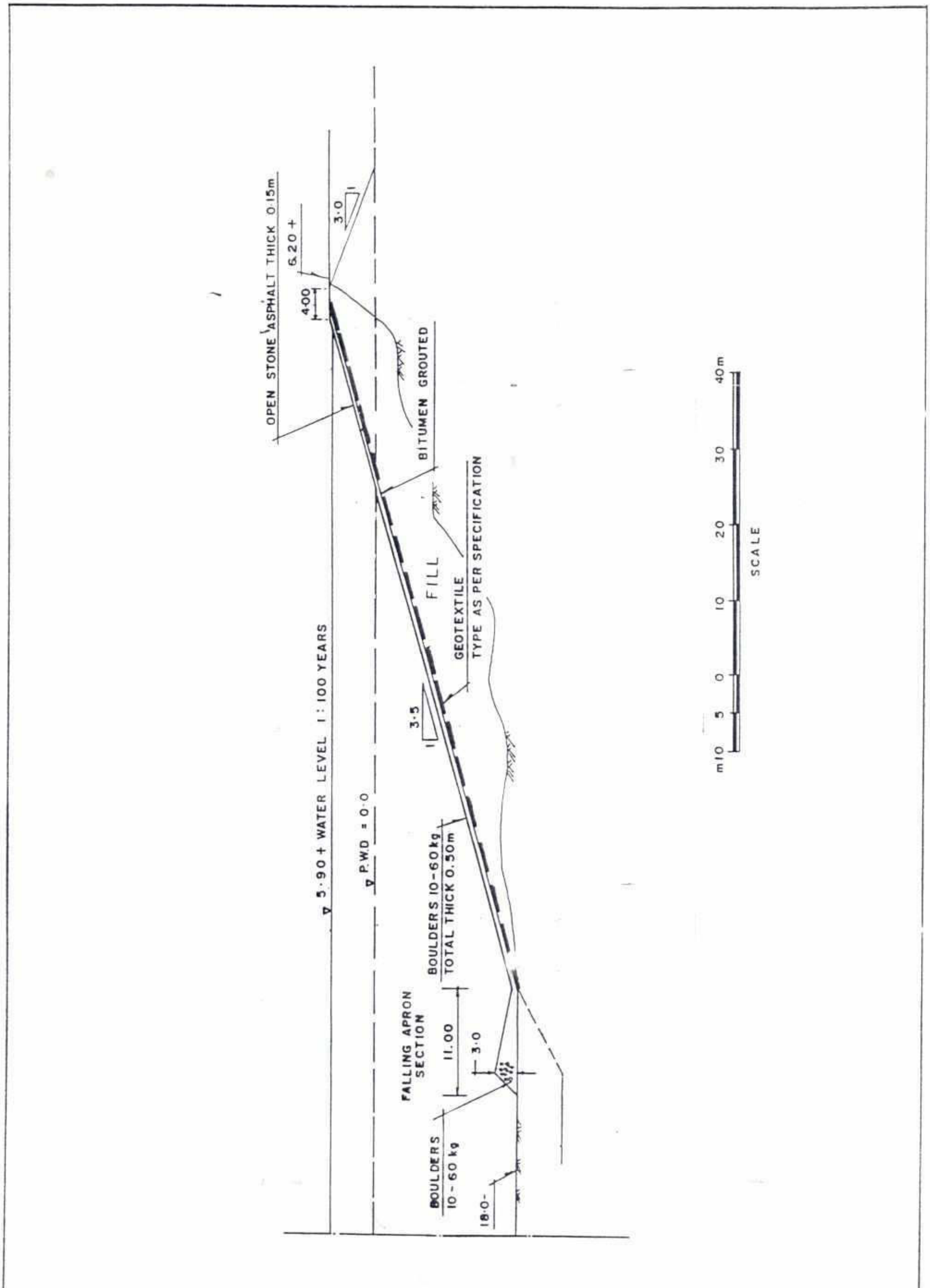
TYPICAL FASCINE MATTRESS	FIGURE G.2.5.1



DETAIL OPEN STONE ASPHALT	FIGURE G.2.5.2



TYPICAL CROSS SECTION GROYNE EKLASHPUR	FIGURE G.2.5.3



TYPICAL CROSS SECTION GUIDE BUND EKLASHPUR	FIGURE G.2.5.4

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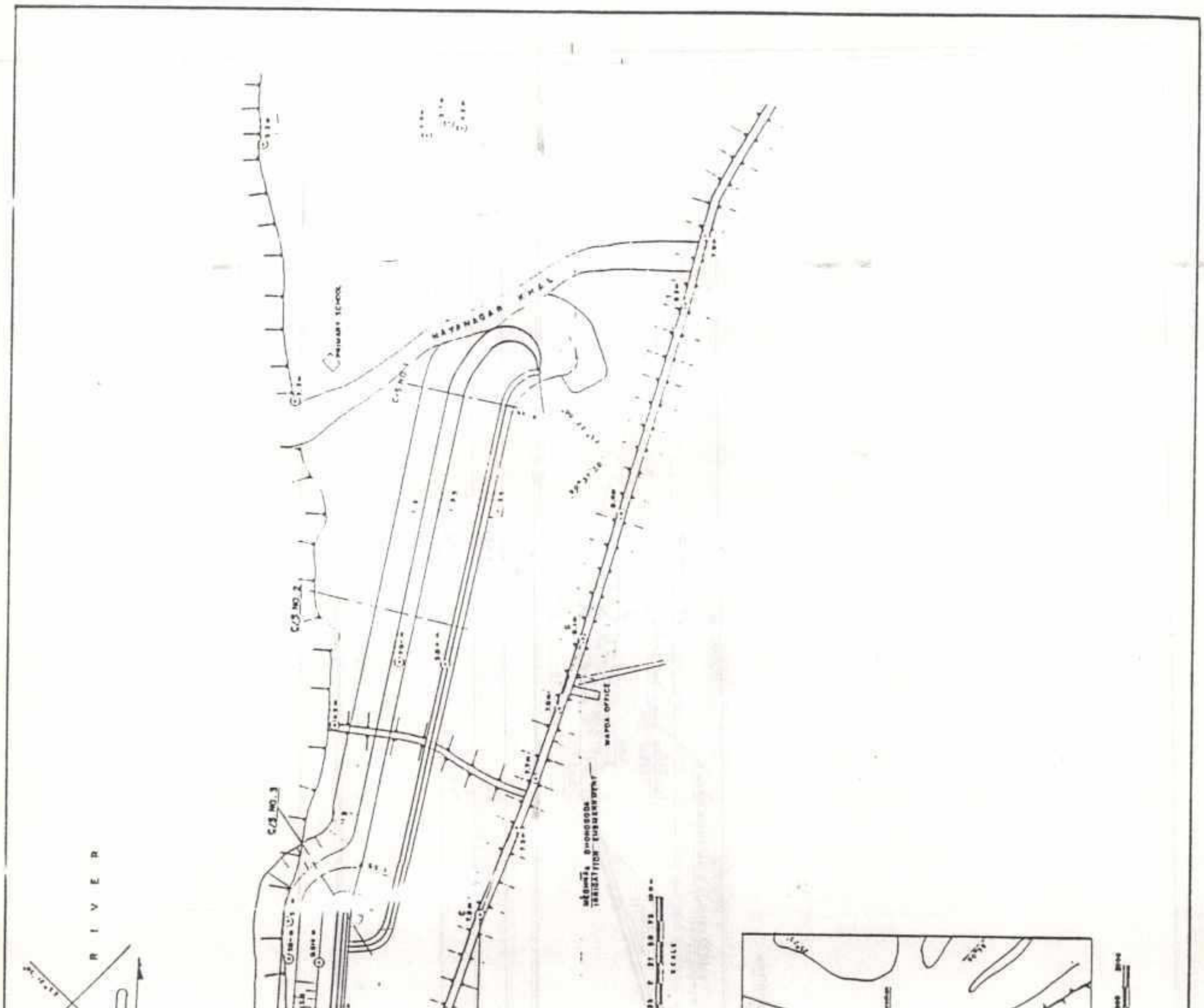
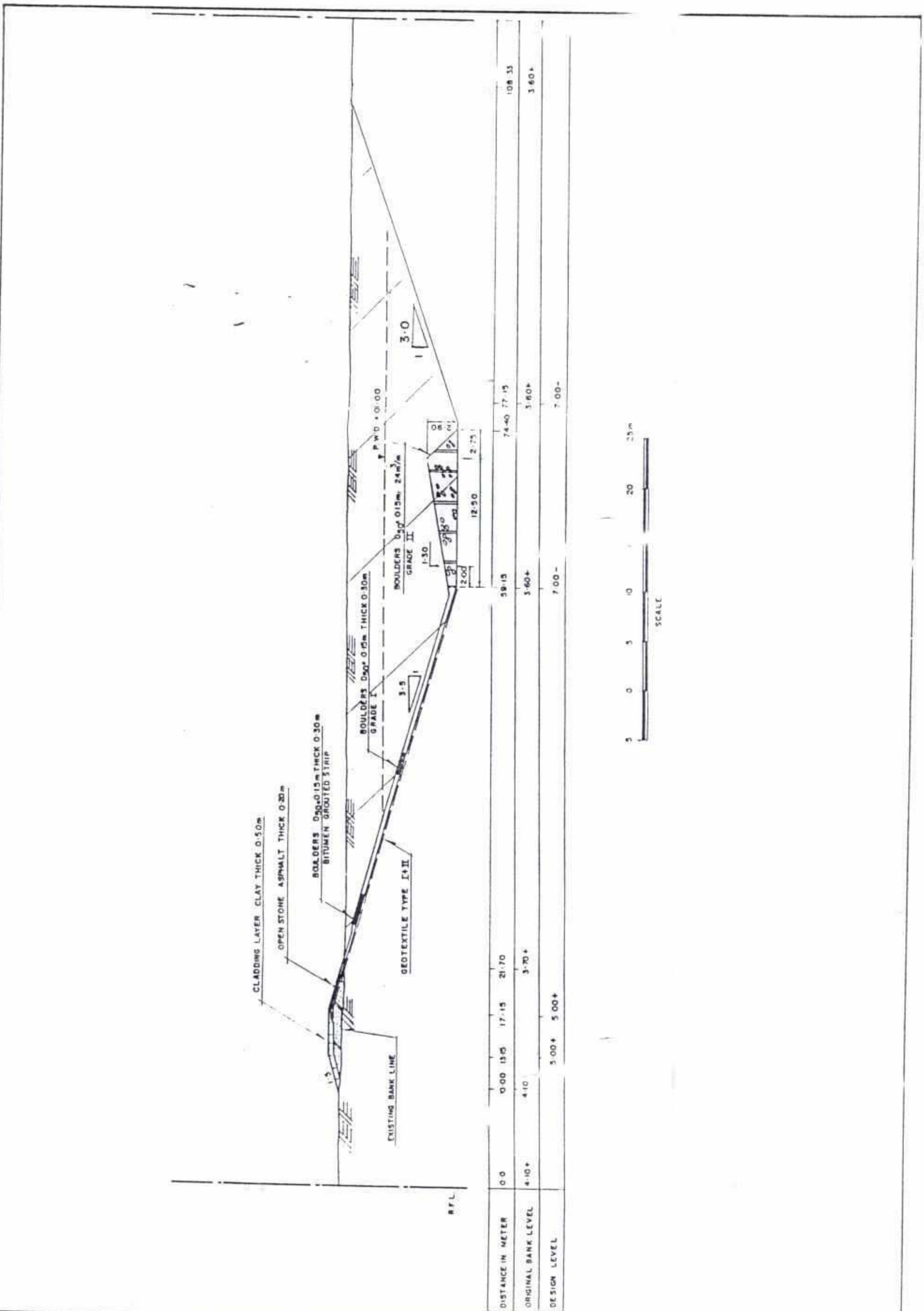


LAYOUT EKLASHPUR	FIGURE G.2.5.5



TYPICAL CROSS SECTION EKLASHPUR

FIGURE .G.2.5.6

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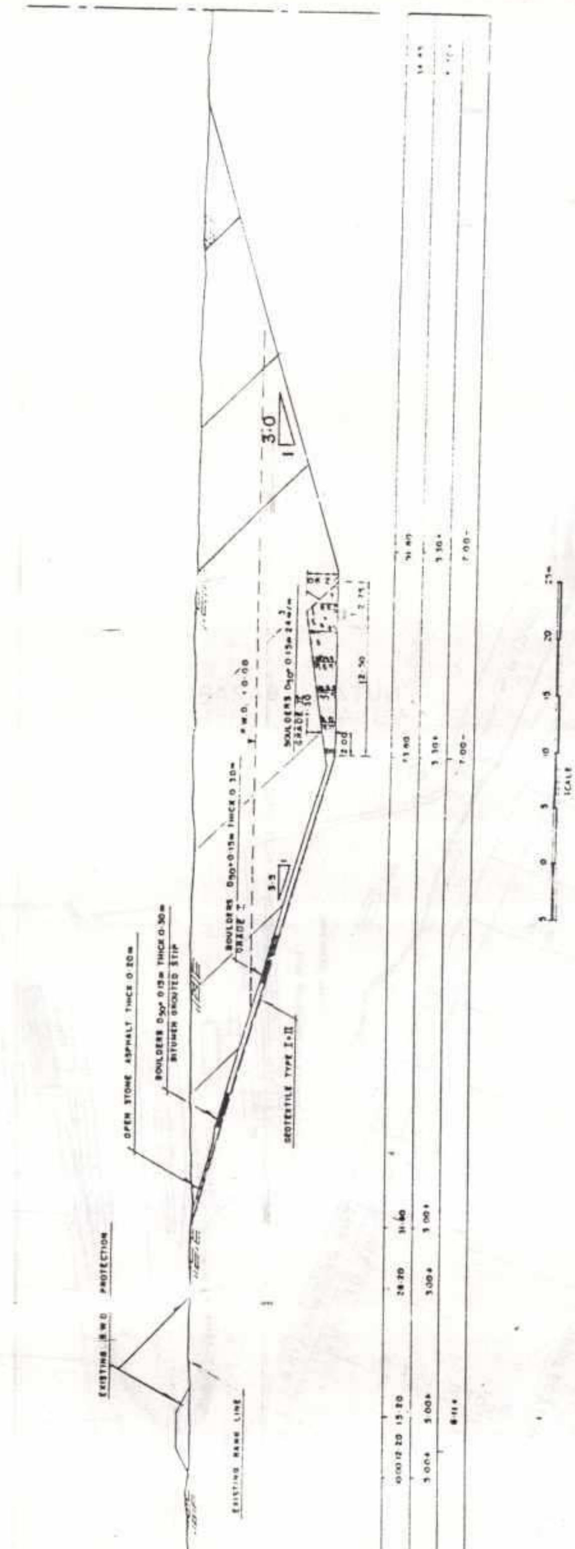
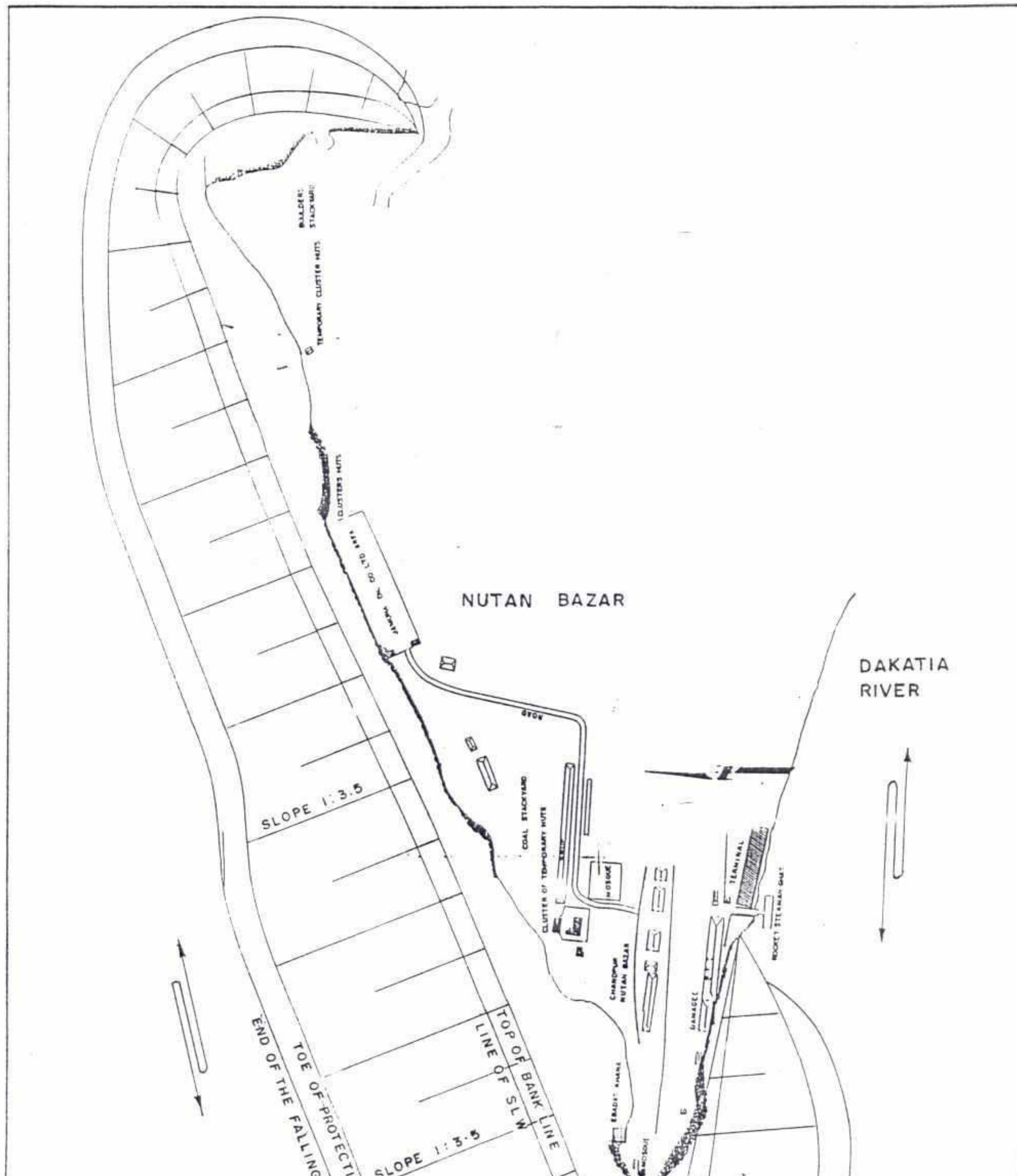


TYPICAL CROSS SECTION EKLASHPUR
(EXTENSION B W D B PROTECTION)

FIGURE G.2.5.7



LAYOUT ADVANCED PROTECTION CHANDPUR

FIGURE G.2.5.8

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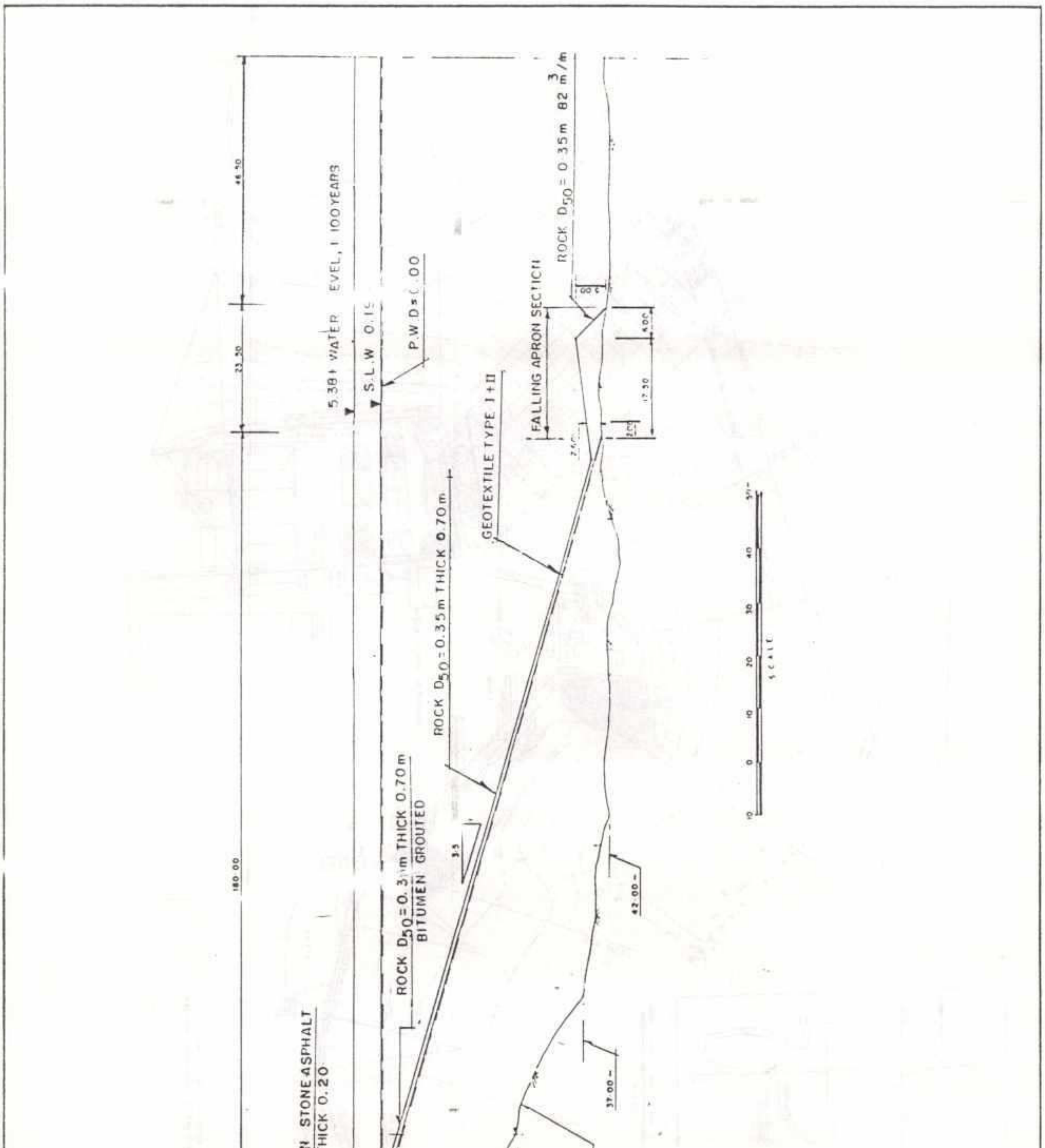
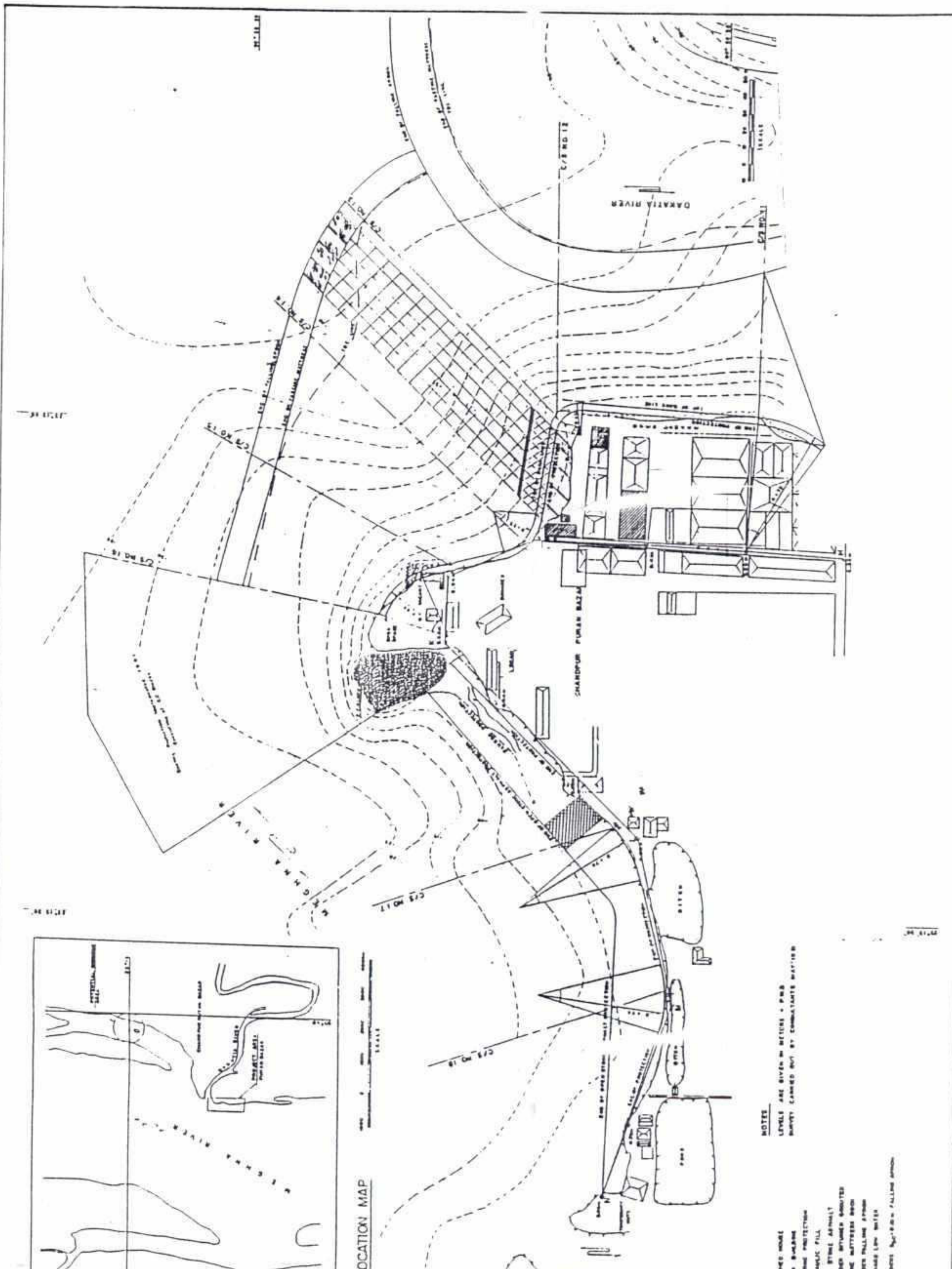
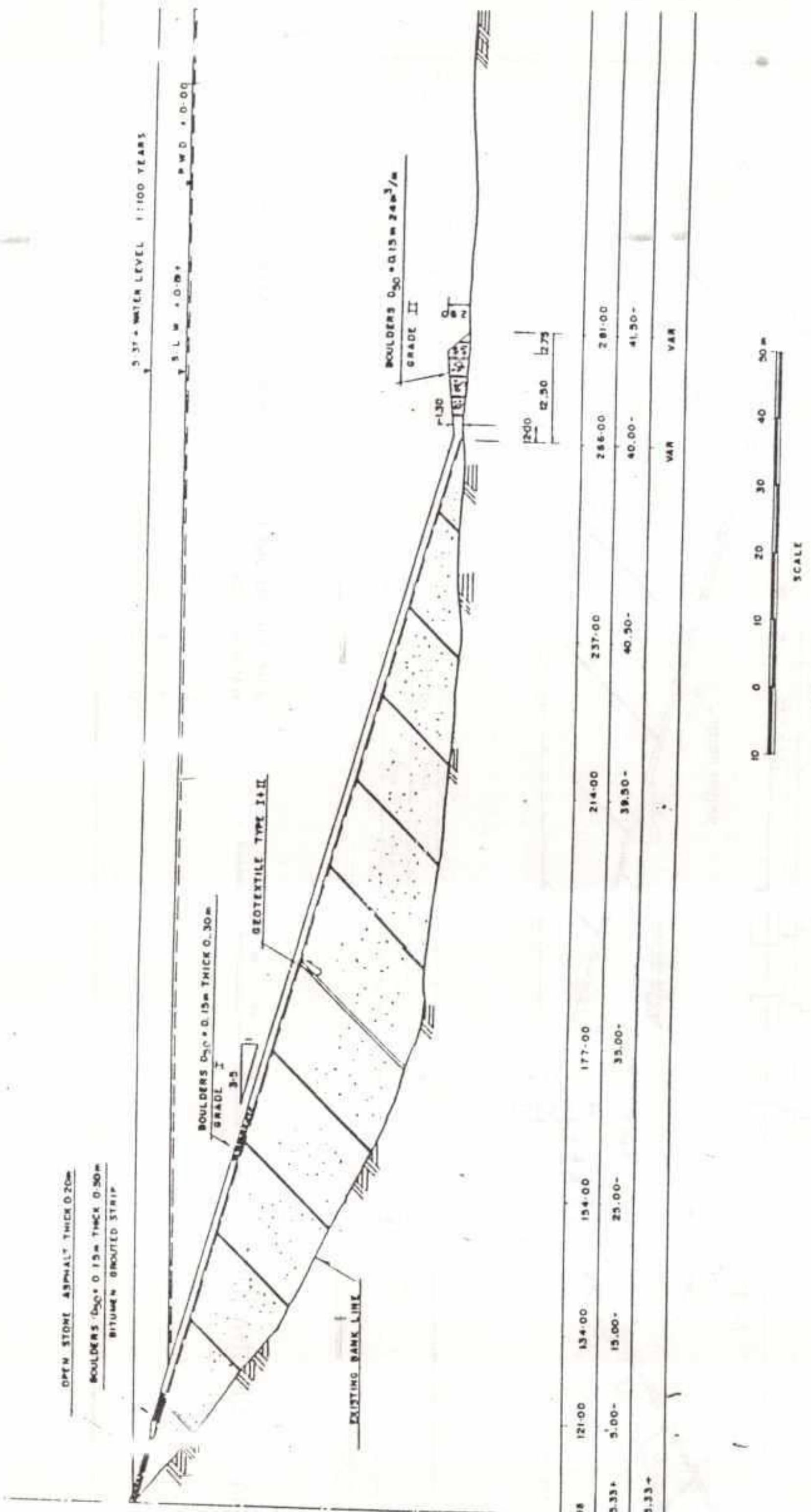


TYPICAL CROSS SECTION CHANDPUR	FIGURE G.2.5.9

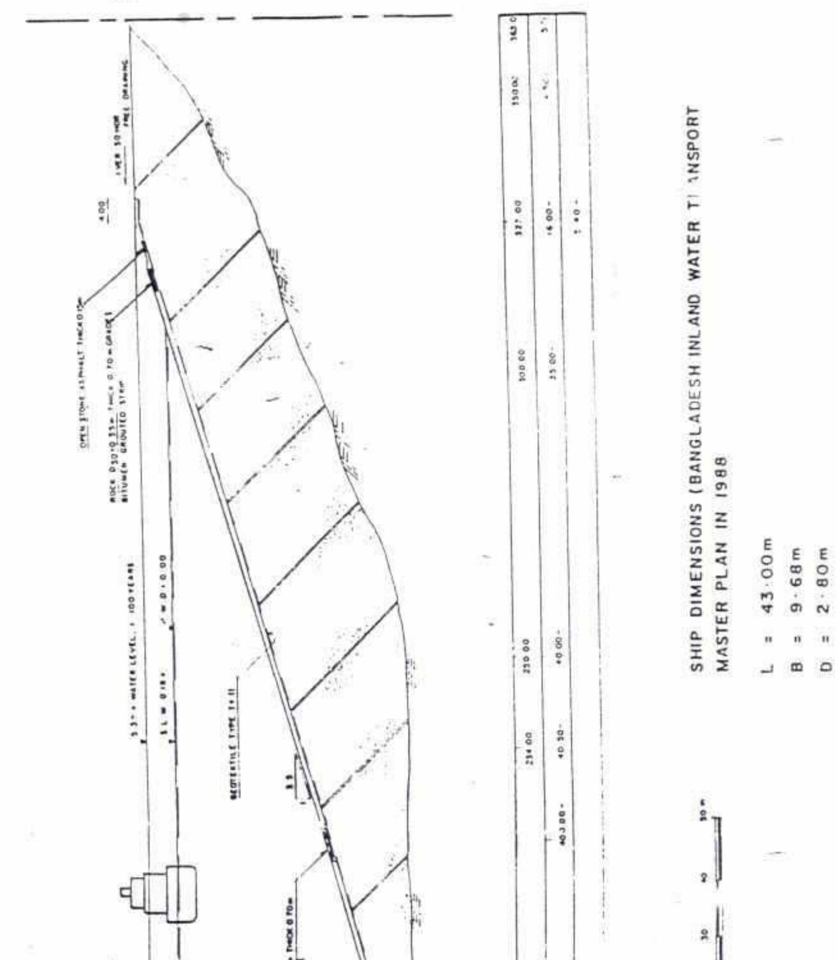


LAYOUT CHANDPUR, PURAN BAZAR

FIGURE. G.2.5.10



TYPICAL CROSS SECTION CHANDPUR, PURAN BAZAR | FIGURE G.2.5.11



TYPICAL CROSS SECTION DAKATIA RIVER

FIGURE. G. 2.5 .12

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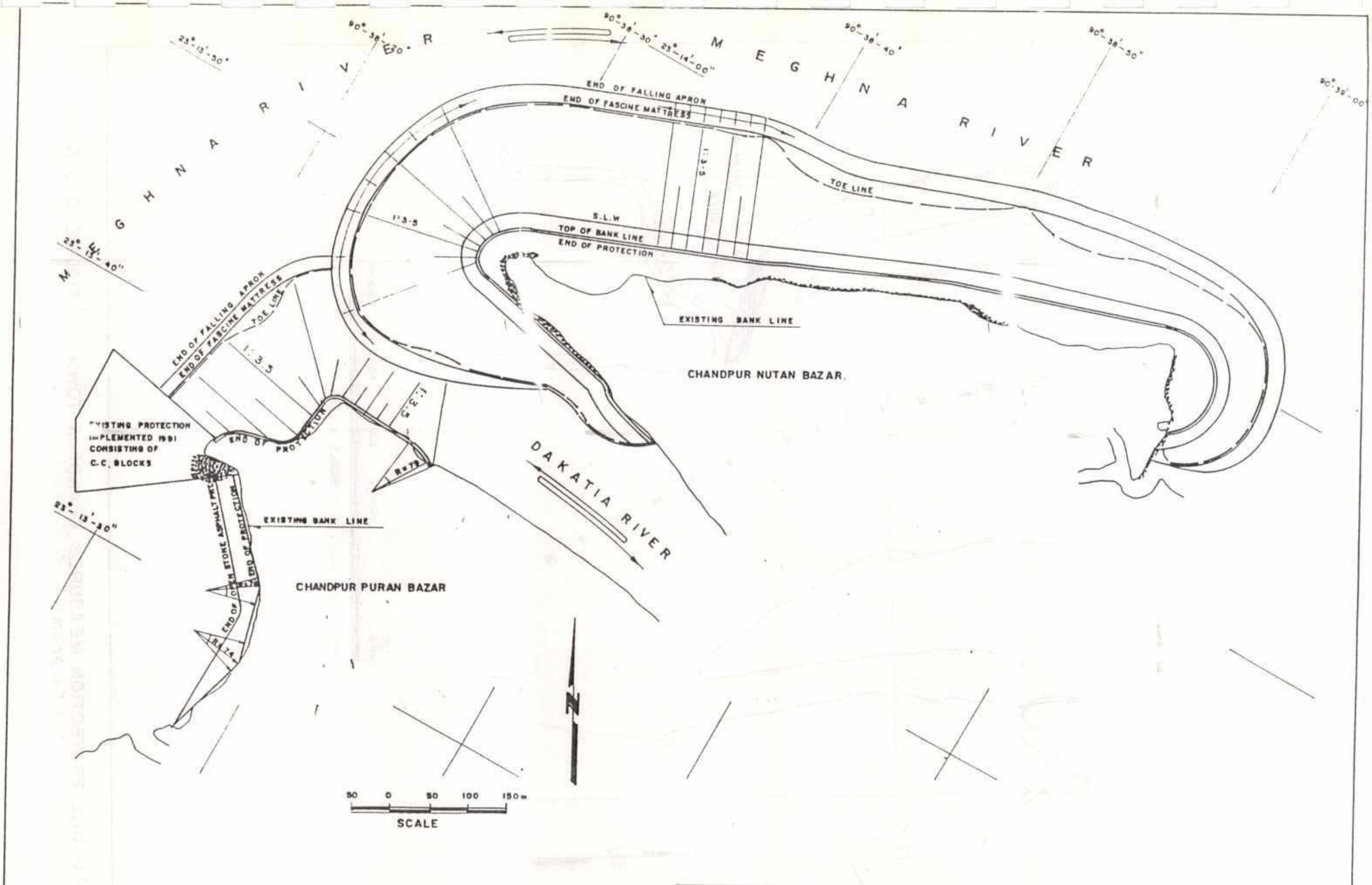

LAYOUT PLAN SHORT TERM PROTECTION WORKS
CHANDPUR TOWN

FIGURE . G.2.5.13

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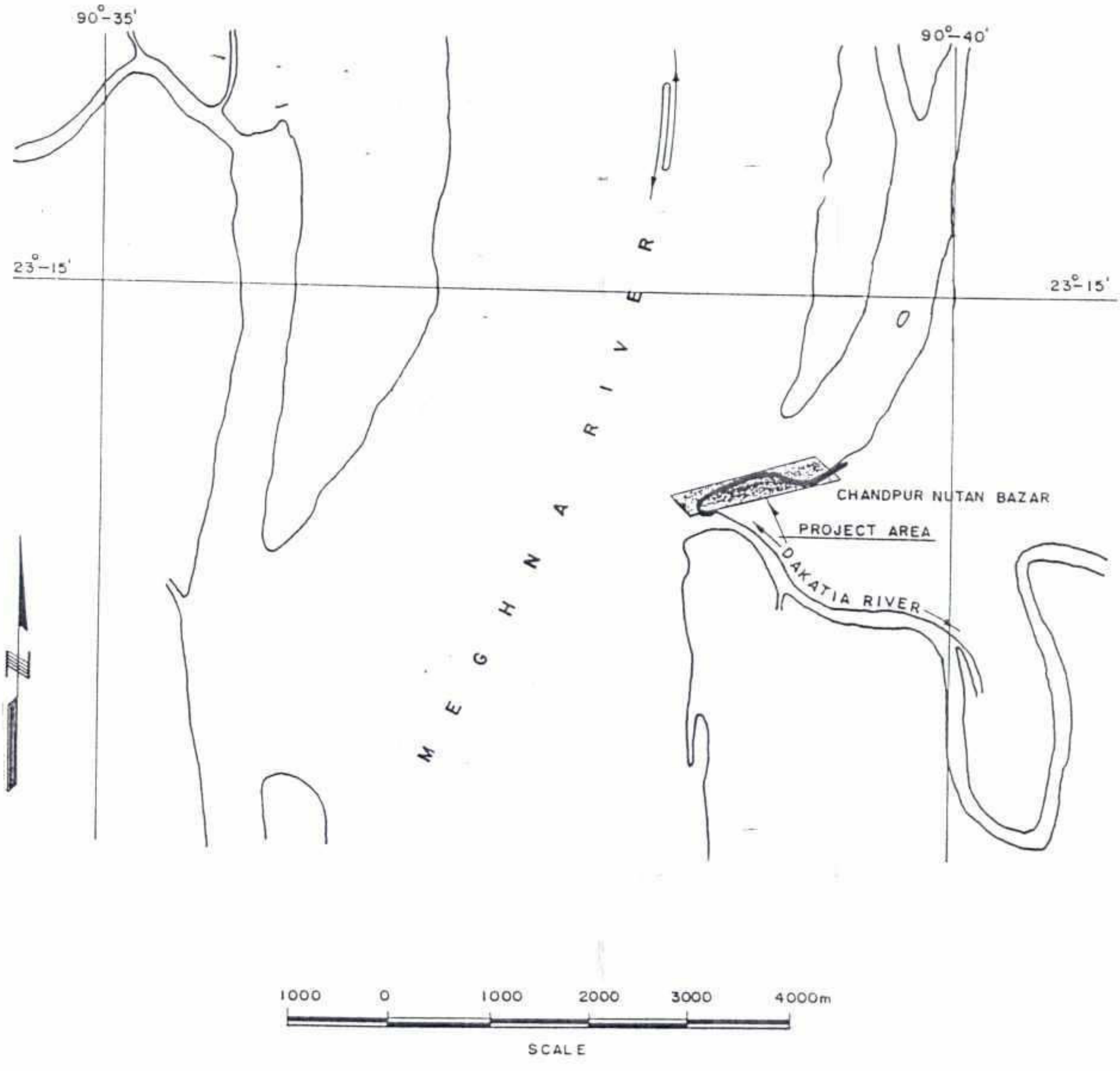

FUTURE PROTECTION MEASURES CHANDPUR TOWN (2003)	FIGURE. G. 2.5.14

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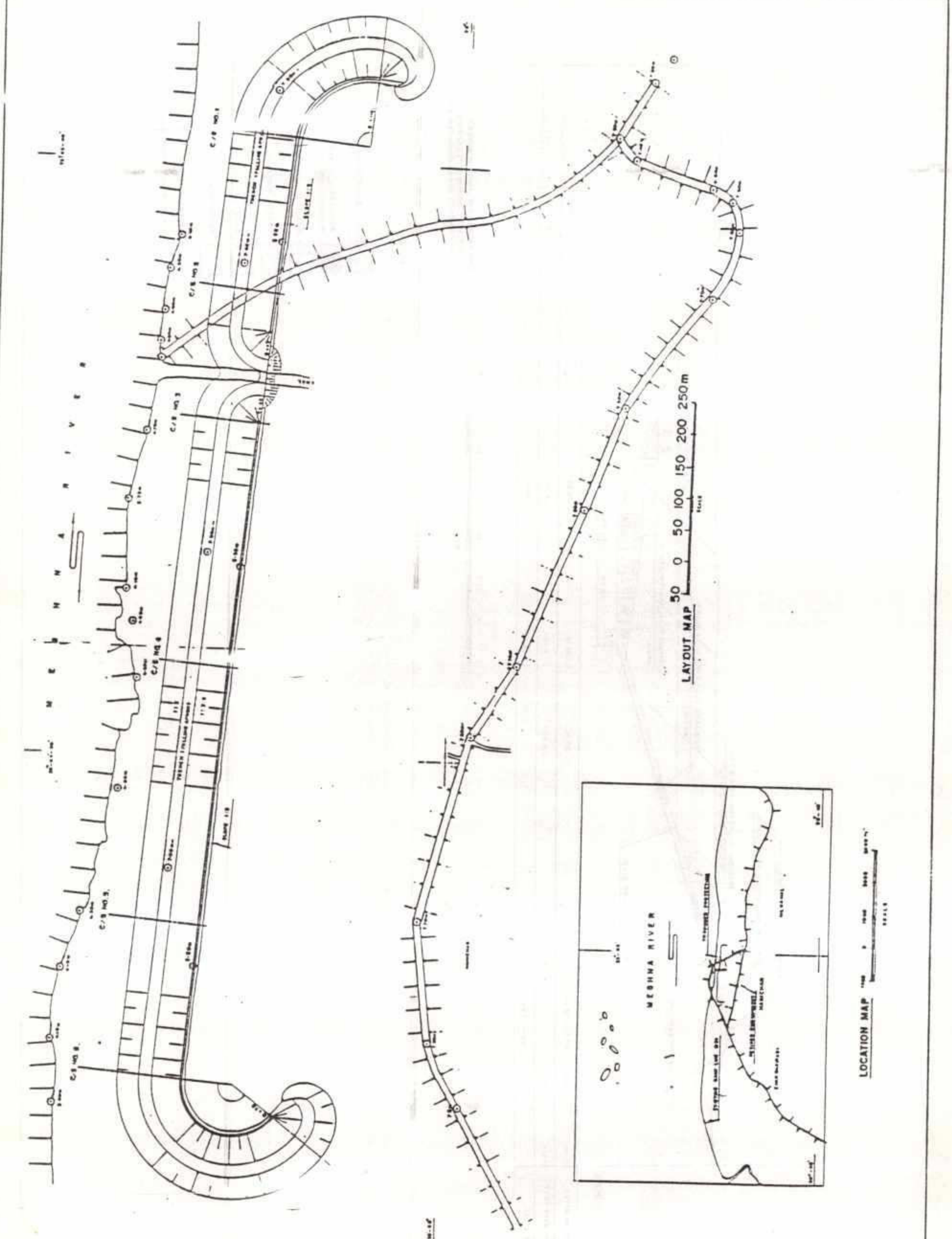
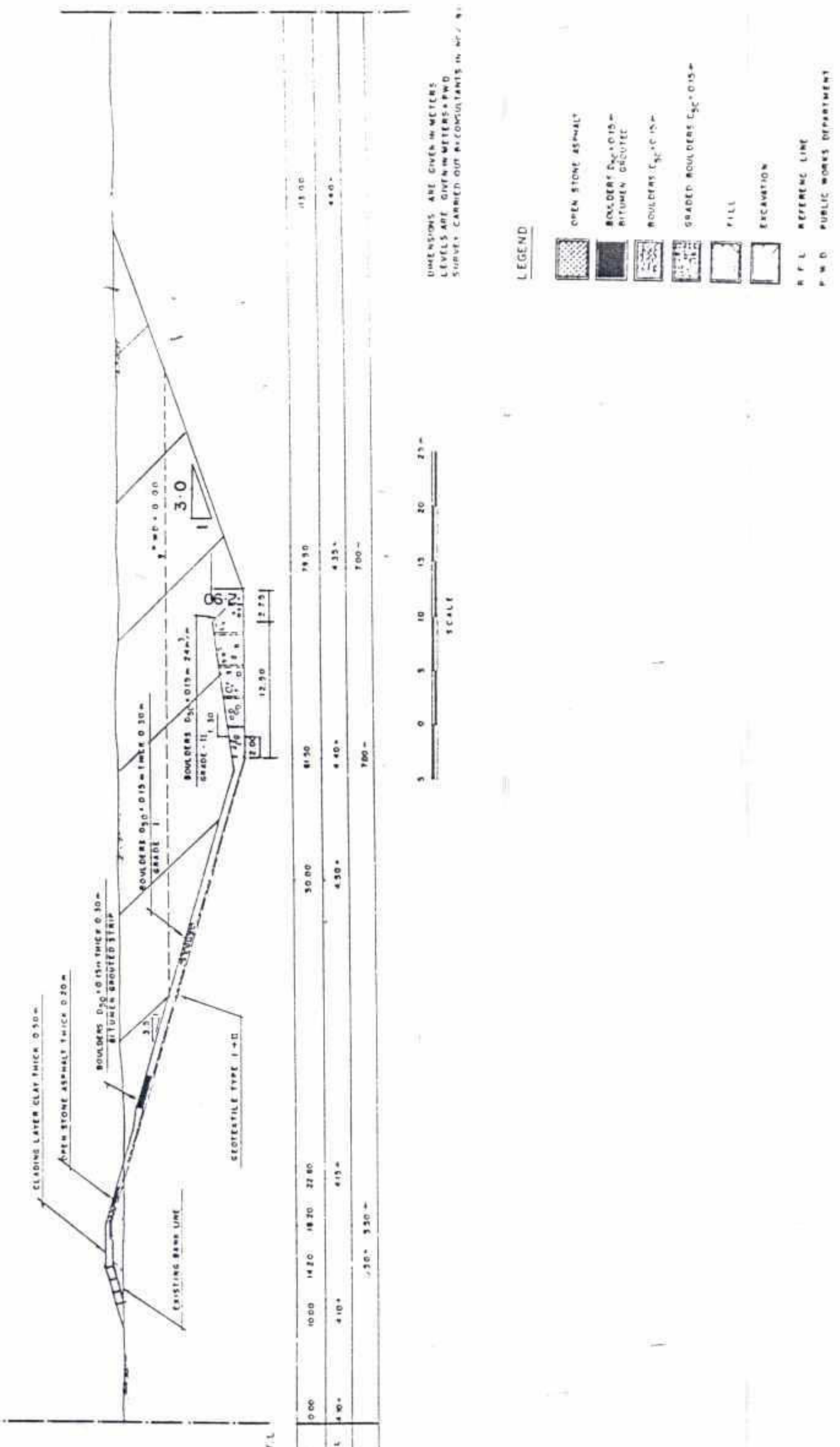


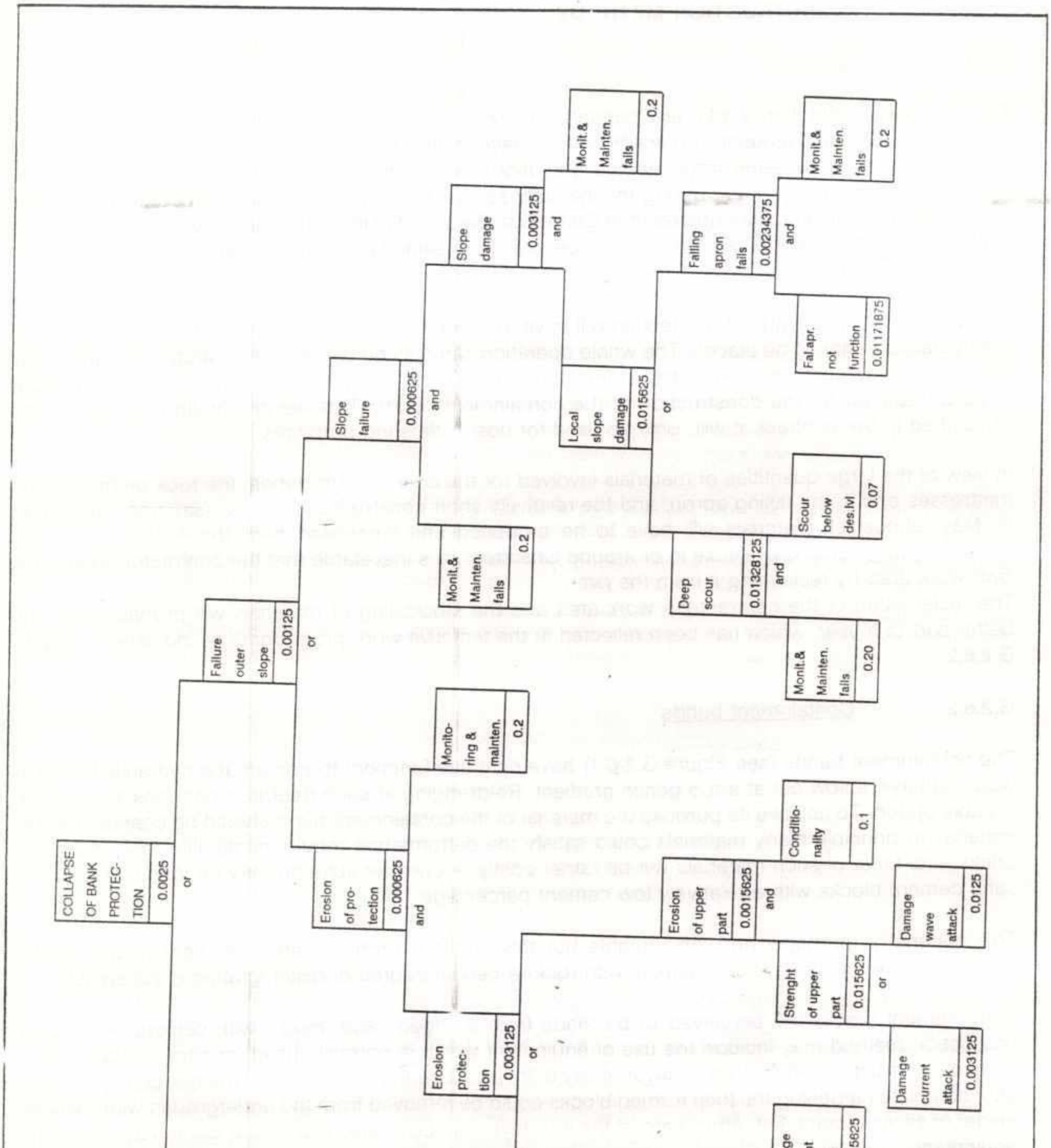
LAYOUT HAIMCHAR

FIGURE G.2.5.15



TYPICAL CROSS SECTION HAIMCHAR

FIGURE G.2.5.16



FAULT TREE CHANDPUR

FIGURE G.2.5.17

G.2.6. CONSTRUCTION METHODS

G.2.6.1 General

The proposed protection works at Chandpur are rather unique. To the Consultant's knowledge no reclamation and subsequent protection works have ever been carried out in an alluvial riverine environment in which such large depths and high current velocities occur. However, the current velocities are not always high: during the low flow season the Lower Mehgna is predominantly a tidal river. Current reversal occurs upstream of Chandpur. So by definition four times every day there will be a slack water period which can, and should, be used for making fast progress with the works. Moreover the maximum current velocities in the dry season do not exceed 0.4 m/s.

Reclamation of the advanced[7] protection will have to be done [illegible] containment bunds, behind whic[illegible] the hydraulic fill has to be placed. The whole operation can only be carried out in stages. The thickness of each layer will probably be of the order of 3m, in or[illegible]r to reduce the quantity of coarse granular material required for the construction of the containment bunds. This height should however not be prescribed in the contract; it will only be used for cost estimating purposes.

In view of the large quantities of materials involved for the containment bunds, the rock on the fascine mattresses and in the falling apron, and the relatively short construction window (say from November till May) almost all materials will have to be produced and stockpiled near the works site. As no sufficiently large area is available in or around Chandpur, it is inevetable that the contractor creates his own work area by reclaiming it from the river.
The reclamation of the contractor's work area and the stockpiling of materials will probably take the better part of a year, which has been reflected in the tentative work programme as indicated in Figure G.2.6.2.

G.2.6.2 Containment bunds

The containment bunds (see Figure G.2.6.1) have only one function: to contain the hydraulic fill which would otherwise flow out at a too gentle gradient. Re-dredging at such depths is not considered to be a viable option. To achieve its purpose the material of the containment bund should be coarse granular material. In principle many materials could satisfy the performance requirements, like rock, boulders, bricks, etc. Most of such materials will be rather costly. A cheaper solution may be found when using sand cement blocks with a relatively low cement percentage.

This material is probably not very durable but this is not a problem: after the construction of the protective revetments (fascine mattress with rock) a certain degree of disintegration is acceptable.

Sand cement blocks are perceived to be made from dredged sand, mixed with cement. A possible production method may include the use of equipment which is normally used for cement stabilisation in road construction. After mixing a layer of say 0.3m and a degree of hardening the mix can be cut and after complete hardening the then formed blocks could be removed from the underground with a wheel-loader or similar equipment. After removal the next layer can be treated in a similar fashion by the mixing equipment.

The contractor should however be free to select his own type of materials for the containment bunds which satisfy the requirements.

For placing the sand cement blocks, or other materials, in the containment bunds, ships or a pontoon equipped with a fall pipe will be required. At the end of the fall pipe a so-called "remotely operated vehicle" is attached, which "holds" the end of the fall pipe into position. Other methods, like the use of a stone dump barge, will probably not lead to a sufficient accuracy in view of the large water depths.

---

[7] Again it is emphasized that 'advanced' refers in principle to position only and not necessarily to advanced techniques though such techniques are indispensable in the case of Chandpur (contr[illegible] Bazar and Munshiganj).

Fall pipe ships are frequently used in the offshore industry for covering up of pipe lines on the seabed with rock. The depth at which such fall pipes are used is up to 100m. For Chandpur the use of a ship would appear to be too costly. Instead one or more pontoons with a fall pipe may be used. The supply of rock to the funnel of the fall pipe could be by towed barges, which are loaded at the work area cum stockyard.

The same equipment can be used for the placing of rock in the falling aprons.

G.2.6.3 Dredging and reclamation

In order to reduce losses of fill during placing as much as possible, it is attractive to use the coarsest possible sand, though at Chandpur one is not likely to find sand with D_{50} > 0.200mm. The best source may be many kilometres from Chandpur. As there is no need for later re-dredging it is opportune to assume that a training suction hopper dredger, with self discharging provisions, is used, rather than a suction or cutter suction dredger. A trailing hopper dredger also diminishes hindrance to navigation on the Lower Meghna. Alternatively, suction dredgers and barges may be used.

G.2.6.4 Fascine mattresses with rock

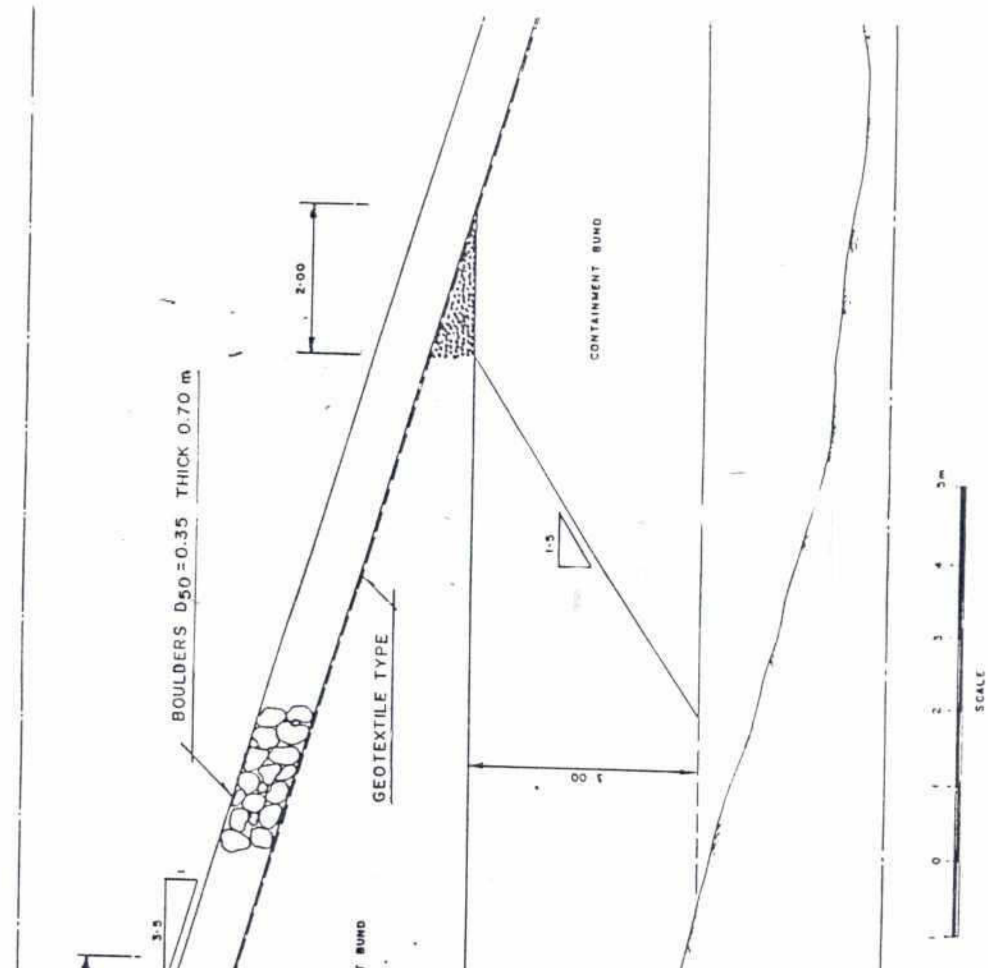
The slope protection mattresses consist of a geotextile fabric with a cover of rock. In view of the absence of a sufficient quantity of large sized boulders in Bangladesh, rock has been proposed. Other alternatives, like the use of concrete blocks, have extensively been researched for the Jamuna Bridge project, but a cover of rock had advantages, including costs, over other alternatives. For estimating purposes it will be assumed that rock will be imported from Malaysia, where a multitude of suitable quarries exist from where the rock could be obtained. Other sources of rock need however not be excluded.

For bringing the geotextile in place it will be necessary to prepare mattresses on a launching ramp. Bamboo fascines have to be fixed to the geotextile to arrive at sufficient buoyancy (necessary during transport) and flexibility, without folding, etc. during the sinking of the mattress. Contrary to the river bank protection works at Bhairab Bazar and Munshiganj, for mattress sinking operations in Chandpur, sophisticated equipment and positioning methods are indispensable. The use of computer controlled stone dumping barges is likely to lead to the desired result. If necessary this can be combined with fall pipe equipment should it appear that curtained sections of the mattresses had not received a sufficient cover. The same equipment can be used for the placing of rock in the falling aprons.

An alternative for the bamboo could be the use of the residual of jute, which is available in widely available in Bangladesh.

G.2.6.5 Construction program

The construction program for the advanced protection at Chandpur Nutan Bazar is presented in Figure G.2.6.2.



DETAIL CONTAINMENT BUND CHANDPUR

FIGURE G.2.6.1

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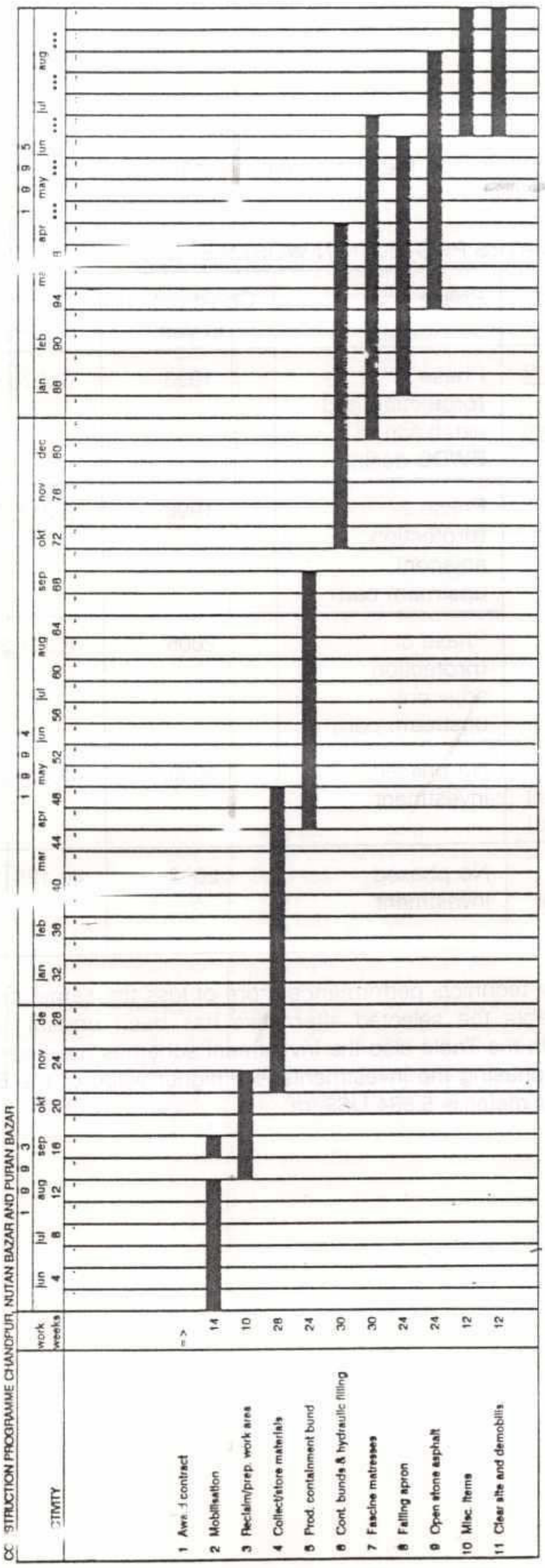

CONSTRUCTION PROGRAM CHANDPUR	FIGURE G.2.6.2

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G.2.7. **COST ESTIMATE**

G.2.7.1 Cost

G.2.7.1.1 Eklashpur

Cost estimates have been made for the most realistic alternatives at Eklashpur site. They are summarised in the following Table.

Table G.2.7.1 COST ESTIMATES PROTECTION WORKS EKLASHPUR

Item	Alternative	Phase	Construction in year	Cost in US$	Total cost in US$
i	Protection of existing embankment	Phase 1 (protection and extension of BWDB design)	1993	10,078,510	26,502,036
		Phase 2 (protection adjacent upstream part)	1998	8,211,763	
		Phase 3 (protection adjacent upstream part)	2005	8,211,763	
ii	Retired embankment (guide bund)	No phased investment	2003	42,154,950	42,154,950
iii	Groyne upstream of Eklashpur	No phased investment	2003	40,185,745	40,185,745

The alternatives are in view of technical performance more or less the same; cost is in this matter the selection criterion. In the Table the selected alternative has been underlined (see Economics of Protection Works, Annex F). In the Table also the investment schemes have been presented (see also Annex F). The advantage of phasing the investments is a higher value of the EIRR. For the selected Alternative the cost per limear meter is 5,684 US$/m.

G.2.7.1.2 Chandpur

The estimated cost of the construction of the protection works at Chandpur is US$ 71.21 million.

A breakdown is presented in the following Table.

Table G.2.7.2 COST ESTIMATE PROTECTION WORKS CHANDPUR, NUTAN BAZAR AND PURAN BAZAR

Item	Quantity	Unit	Unit cost in US$	Cost in US$
1 Dredging	3,388,000	m^3	3.58	12,142,106
2 Working area/materials	1	-	337,216	337,216
3 Earthworks above SLW	1	-	473,040	473,040
4 Clear site and reinstate	1	-	236,520	236,520
5 Open stone asphalt	41,800	m^2	30.93	1,292,797
6 Fascine mattress rock	164,600	m^2	48.03	7,905,255
7 Fascine mattress boulders	25,375	m^2	17.48	403,530
8 Falling apron	98,400	m^3	69.73	6,861,106
9 Grouting of rock and boulders	1,865	m	62.58	116,716
10 Containment bunds	372,000	m	35.45	13,187,577
11 Contractors cost and supervision	1	m^3	1,637,200	1,647,200
12 MOB and DEMOB (excl.dredging equipment)	1	-	88,000	88,000
13 MOB and DEMOB dredging equipment	2	-	2,281,578	4,563,157
				======
				49,284,220
14 Physical contingencies	15	%		7,392,633
15 Contractors margins and fees	22	%		10,834,528
16 Engineering and supervision	7.5	%		3,696,317
				======
TOTAL				71,215,698

The cost per linear meter for the short term protection works at Chandpur are 38,185 US$/m.

To ascertain the sustainability of these protection works at Chandpur Town protective measures are required in future. In the following Table the cost estimates and the investment scheme of these measures are presented [illegible] the protection of Chandpur Town is included.

Table G.2..7.3 INVESTMENT SCHEME CHANDPUR

Item	Description of protection	Year of investment	Cost in US$
1	Protection of Chandpur Town	1993	71,215,698
2	Protection of outflanking area (L=400m)	2003	7,500,000

G.2.8 ANALYSIS AND SELECTION OF ALTERNATIVES

G.2.8.1 Chandpur Town Protection

In view of advantages and disadvantages mentioned earlier and the cost of the various alternatives Consultants selected the advanced protection at Nutan Bazar/Puran Bazar.

G.2.8.2 Eklashpur

As mentioned before the alternative of protecting the existing embankment has been selected. This protection will partly be build in the dry and will also integrate the BWDB protection now under construction. For economic reasons, the construction will be carried out in three phases over a period of approximately 20 years.

G.2.8.3 Haimchar

Also for the site of Haimchar it has been already mentioned that the alternative of a protection, shaped like a guide bund has been selected. Also some of the existing embankment should be replaced. For economic reasons, the construction will be carried out in three phases over a period of approximately 20 years.

G.2.7.1.3 Haimchar

Cost estimates have been made for the most realistic alternatives at Haimchar site. These are summarised in the following Table.

Table G.2.7.4 COST ESTIMATE PROTECTION WORKS HAIMCHAR

Item	Alternative	Phase	Construction in year	Cost in US$	Total cost in US$
i	guiding protection	protection (length=3,800)	1993	18,371,682	18,371,682
		replace embankment (length=6,500m)	2008		
ii	guiding *) protection	protection (length=2,800m)	1993	12,990,712	23,210,329
		protection (length=1,000m)	2008	10,219,617	
		replace embankment (length=6,500m)	2008		
iii	guiding *) protection	protection (length=1,800m)	1993	10,923,482	29,320,365
		protection (length=1,000m)	1998	8,012,504	
		protection (length=1,000m)	2008	10,384,379	
		replace embankment (length=6,500m)	2008		
iv	protection of existing embankment **)	protection (length=2,800m)	1993	12,990,712	25,981,424
		protection (length=2,800m)	2008	12,990,712	

*) guiding protection = the short protection shaped like a guide bund protecting partly the existing embankment and replacement of embankment;

**) protection of the existing embankment = as indicated [illegible] without replacement of embankment.

The alternatives are in view of technical performance more or less the same: cost is in this matter the selection criterion. In the Table the selected alternative has been underlined (see Economics of Protection Works, Annex F). In the Table also the investment schemes have been presented (see also Annex F). The advantage of phasing the investments is a higher value of the EIRR.

G.2.7.2 Maintenance

For maintenance reference is made to Chapter G.1 of this Annex and to the Main Report.

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G.2.9 CONSIDERATIONS ON FUTURE DEVELOPMENT OF LOWER MEGHNA

G.2.9.1 General

In the previous sections protection works have been proposed for the sites of Eklashpur, Chandpur and Haimchar only. The proposed works are short term measures for protection of the sites which are to be integrated into long term measures. In this section some considerations are presented on possible long term measures for the Lower Meghna.

Firstly, the proposed protection works for Eklashpur, Chandpur and Haimchar will be considered as a 'package' of investments; which is contrary to the approach presented in previous sections. Thereafter some possible long term measures are presented in a very general manner. These measures require further detail studies of the Lower Meghna.

G.2.9.2 Protection of Eklashpur, Chandpur and Haimchar

Protection works now proposed for short term measures are not conflicting with each other and have been selected in respect of future geo-morphological developments of the Lower Meghna. Nevertheless, in respect of cost, they have been considered as 'sites on it self' without looking into integrated cost schemes. In the following Table the total investment scheme for the protection works has been presented.

Table G.2.9.1 INVESTMENT SCHEME LOWER MEGHNA FOR PROTECTION WORKS AT EKLASHPUR, CHANDPUR AND HAIMCHAR

Construction of protection	Year of investment	Cost in US$
Chandpur Town	1993	71,215,698
Haimchar (first part)	1993	10,923,482
Eklashpur (first part)	1993	10,078,510
Eklashpur (second part)	1998	8,211,763
Haimchar (second part)	1998	8,012,504
Chandpur (length=400m)	2003	7,500,000
Eklashpur (third part)	2005	8,211,763
Haimchar (third part)	2008	10,384,379
	TOTAL cost in US$	134,538,099

In the Economics of Protection Works, Annex F, this total investment scheme has been analysed; benefits are to be considered only in the impact areas of the protection works.

G.2.9.3 Possible long term measures Lower Meghna

The protection works at the three sites are acting as hard points in the Lower Meghna; they protect the areas of interest at these specific sites. Irrigation areas at the left bank, situated between Eklashpur and Chandpur and Chandpur and Haimchar, are not protected, since they are beyond the impact area of these protection works (see Figure G.2.5.9.1).

River training of the whole Lower Meghna, and thus protection of all the irrigation areas at the left bank, requires additional protective measures. Looking into (i) the distance between protection works at Eklashpur and Chandpur, (ii) the major characteristic length of the Lower Meghna, which is approximately 5,000m, and (iii) the future geo-morphological development of the Lower Meghna, (see Annex B), it is sufficient to construct a 'hard point' between Eklashpur and Chandpur. The latter could be a T-shaped groyne having a length of approximately 1,000m.

It is emphasized that these long term measures proposed are based on the information now available and will require further detailed studies.

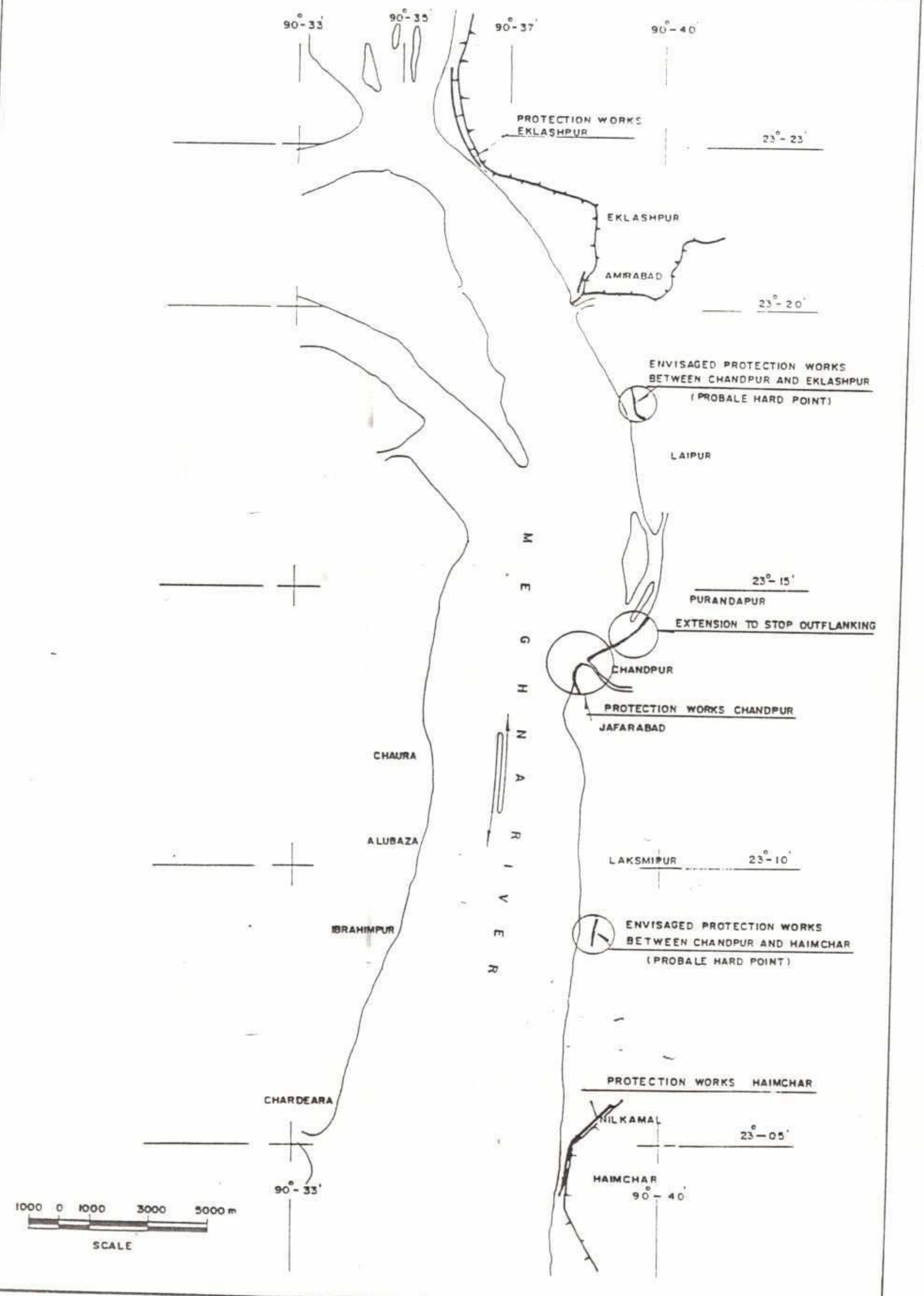
Implementation of all the protection works mentioned increases the area of influence and thus also increases the benefits of all protective measures. This applies to both short term and envisaged long term measures.

In the following Table the investment scheme for river training of [illegible] ...wer Meghna has been presented. The years of investment take into account the geo-morphological development and are accordingly proposed.

Table G.2.9.2 INVESTMENT SCHEME LOWER MEGHNA FOR PROTECTION WORKS AT EKLASHPUR, CHANDPUR AND HAIMCHAR AND ADDITIONAL PROTECTION WORKS LEFT BANK LOWER MEGHNA

Construction of protection	Year of investment	Cost in US$
Chandpur Town	1993	71,215,698
Haimchar (first part)	1993	10,923,482
Eklashpur (first part)	1993	10,078,510
Protection of Eklashpur-Dhonagoda River (pump station)	1993	5,115,736
Eklashpur (second part)	1998	8,211,763
Haimchar (second part)	1998	8,012,504
Hard point between Eklashpur and Chandpur	2002	9,3[illegible],345
Chandpur (length=400m)	2003	7,500,000
Eklashpur (third part)	2005	8,211,763
Haimchar (third part	2008	10,384,379
	TOTAL cost in US$	149,041,180

In the Economics of Protection Works, Annex F, this total investment scheme has been analysed; benefits have been calculated taking into account protected irrigation areas which are situated in the area of impact of the envisaged protection works.



SHORT TERM MEASURES INTEGRATED INTO ENVISAGED LONG TERM MEASURES LOWER MEGHNA (EKLASHPUR TO HAIMCHAR)

FIGURE G.2.5.9.1

2f2

Chapter G.3

EMERGENCY DESIGNS - CHANDPUR

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MEGHNA RIVER BANK PROTECTION

SHORT TERM STUDY

VOLUME V - ANNEX G

CHAPTER G.3. EMERGENCY DESIGNS - CHANDPUR

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G.3.1 INTRODUCTION

In the past the river bank at Chandpur has, with a limited success been protected against erosion from currents and wave attack by means of temporary protection works. It was common practice to construct revetments along the upper part of the eroded bank and in some cases over the complete slope. In the past also a groyne was constructed in the area of Puran Bazar.

Because of the extreme velocities during high water, steep slopes and very deep scour close to the bank, it is difficult to construct an adequate protection applying local materials, traditional methods and local contractors only.

Accordingly, each year the rather steep bank slopes and the shifting of the thalweg results in losses of parts of the temporary protection works, which in some cases have to be reconstructed again on the attacked bank.

Generally speaking, it is always possible to choose between either high investments and relatively low maintenance, or the opposite.

In Chandpur the protection works implemented can in fact be considered as a low investment/high maintenance structure. This means that the river bank protection is classified as a temporary one.

No doubt it is possible to design a structure that can be considered as short-term permanent protection. However, also such a structure requires close monitoring and maintenance. This maintenance consists of two components:

- During peak discharges at some locations damage will occur because prevailing conditions exceeded the design values. (based on probabilistic design concepts, the chance that this occurs can be established and should govern the design)

- The erosion upstream or downstream of the protection works continues and at a certain moment extension of the structure is required to prevent that the current will attack the structure from behind. Also for this purpose careful and systematic monitoring is required.

This type of high investment structures require commitment of the government to ensure its monitoring and the appropriate maintenance.

As Chandpur is considered by GOB as an emergency, designs should be made soonest. Emergency designs for the protection of the most critical areas are, however, generally not the result of an optimisation of functional requirements, costs and benefits.

As discussed in Chapter G.2 the Consultants have already selected and designed short-term measures for the protection of Chandpur. Consequently, the most convenient approach for the layout of the emergency designs, is to design a structure which is part of the proposed short-term measures and which can be integrated into the layout of the short-term measures later on. It means that the investment for protection of Nutan Bazar can be considered as a first stage of the short-term protection works proposed for Chandpur.

G.3.2 EXISTING SITUATION

Consultants have reviewed previous studies and the design and construction reports prepared b[illegible] Halcrow. As to the emergency works constructed last year in Nutan Bazar and Puran Bazar (Category A in the National Committee Report for Protection of Chandpur), the Consultants feel compelled to make the following observations:

(a) The geotextile bags filled with sand, which are supposed to act as a filter, have some serious draw backs. It is Consultants' considered opinion that because of the construction and monitoring methods and practices used, it is wishful thinking to presume that a sufficiently[1] large area of the protected slopes is covered with bags. Moreover between individual bags openings will exist, through which soil particles of the underlying soil can migrate.

(b) The bags were on the average filled with sand which was far too fine, as described in the report on the construction of the emergency measures prepared by Halcrow. Consultants expect therefore that many, if not most, bags are now, or soon will be, virtually empty. Moreover Consultants expect that many bags were punctured by the sharp corners of the concrete blocks during the dumping operation. This would imply that the concrete blocks placed on top of the bags are now resting either on the bare slope or on a piece of filter fabric which will have no connection with surrounding pieces.

Based on the above Consultants fear that at present there is no properly functioning filter under the concrete blocks and that underlying soil is now, during high river flows, migrating in large quantities. Consultants do therefore not expect that the present temporary emergency protection constructed at Chandpur will last very long and fear that it will deteriorate in r[illegible]re than a few years time.

---

1 Say at least 95 to 98 %.

G.3.3 EMERGENCY PROTECTION

G.3.3.1 **General**

Short term measures as now proposed, see Chapter G.2, consist of an advanced protection in front of Nutan Bazar and as such protecting both Nutan Bazar and Puran Bazar. The latter by diverting the flow from the critical area at Puran Bazar.

Emergency measures as now proposed consist partly of the aforementioned advanced protection, viz. the part at the Mosque and the Railway Station (see Figure G.3.3.1)

The area which will be protected by the emergency works cover the Category A and C areas upstream of the confluence of the Dakatia river as mentioned in National Committee Report for protection of Chandpur.

G.3.3.2 **Design concept**

Emergency protection works as carried out until now at Chandpur are based on the concept that local materials and work methods were to be used. Designs were subsequently adapted to local resources. It is however the Consultants opinion that the problems and threats of river erosion at Chandpur, bearing in mind associated phenomena like current velocities, scour depths and earthquake risks, are of such magnitude that they are beyond the realm of solutions using only, or even primarily, local resources.

It therefore stands to reason that providing revetments on the existing slopes can be labelled as a temporary protection with a high risk of failure. There appears to be, however, another option for emergency protection works at Chandpur: provide a protective structure on a safe slope at the river side of the present protection works. Construction of such a stable slope (safety factor = 1.5) can only be achieved by means of an 'advanced' protection.

An advantage of the concept to incorporate emergency works as part of the short-term measures is that money is not wasted and that the emergency designs are useful to protect Puran Bazar.

At present, Nutan Bazar is a critical point due to possible outflanking of the CC-block protection washing out the Railway Station, market etc. while the Meghna River will join the Dakatia River (at the location of the BIWTA terminal). It is noted that between the two rivers only 40 meters of land are left.

Any other measure of protection over the present slopes and without a proper filter underneath has a very limited life (high risk of failure). This will not serve the purpose of protection to the endangered infrastructure. The present practise of protection with geo-textile bags and gabions (filled with boulders) in some areas puts a surcharge on critical slopes. Therefore, failure (slides) of parts of the bank are most likely to occur.

A layout and a typical cross section over the 'advanced' protection with a slope gradient of 1 : 3.5 is given in Figures G.3.1 and G.3.2 respectively.

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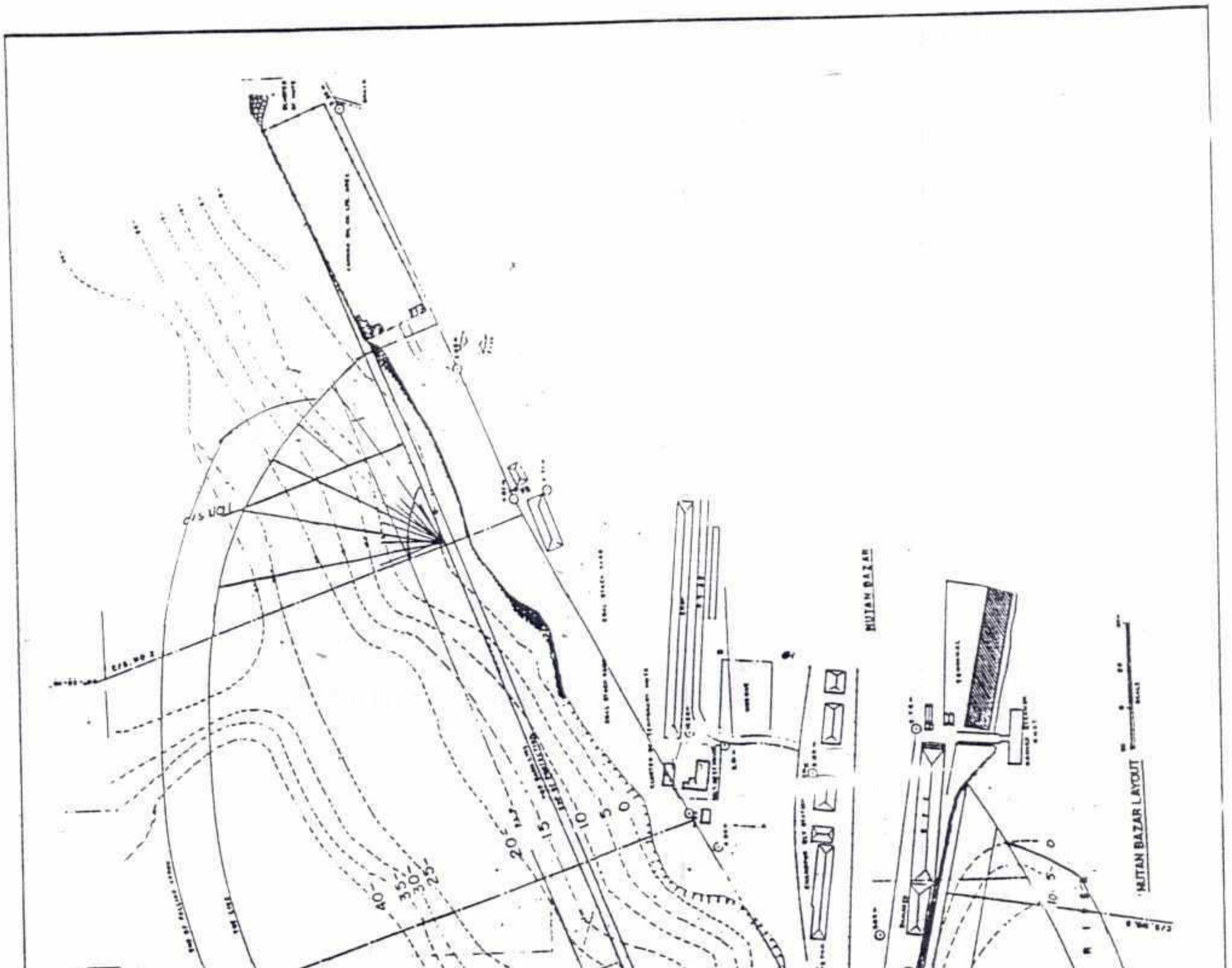
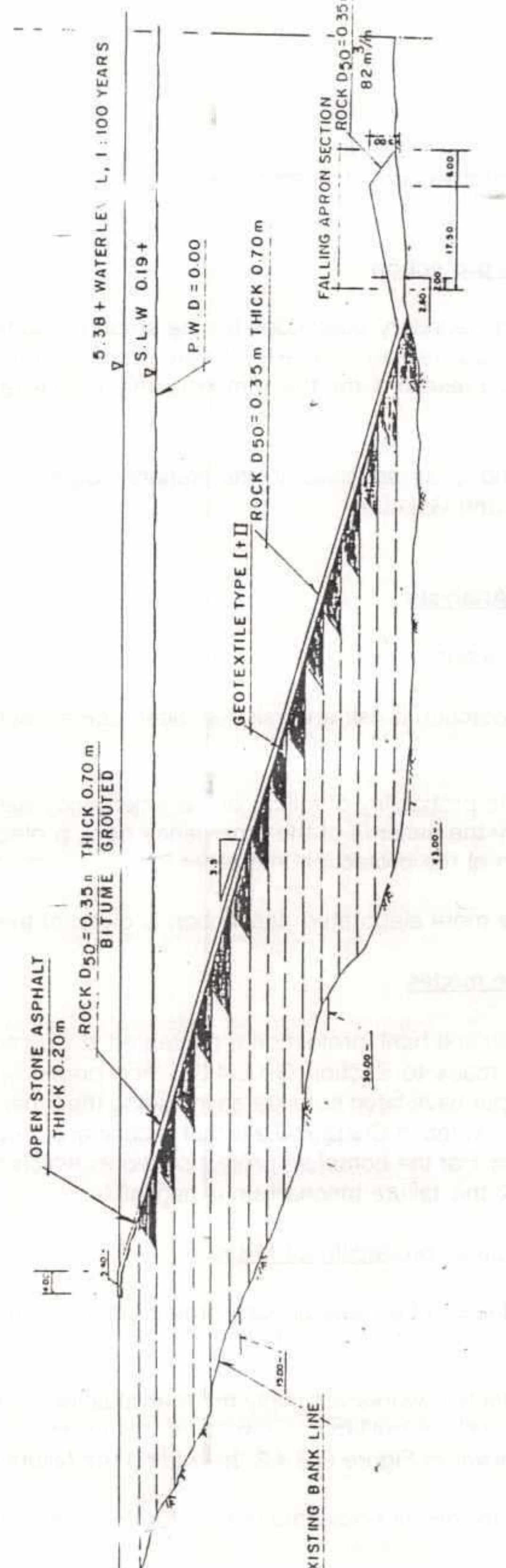


LAYOUT EMERGENCY PROTECTION WORKS CHANDPUR	FIGURE G.3.3.1



TYPICAL CROSS SECTION

FIGURE G.3.3.2

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G.3.4 DESIGN

G.3.4.1 Geotechnical stability

For the emergency protection works the same geotechnical boundary conditions are valid as presented in Chapter G.2 for the short term measures. Therefore the same design considerations are applied here.

G.3.4.2 Slope protection

The design considerations and boundary conditions for the slope protection as presented in Chapter G.2 for the short term measures are also applicable for the emergency protection works. The same holds for the design considerations presented for the dimensioning of the falling apron in relation with expected scour depths.

The layout covers areas A and C as specified in the National Committee Report for Protection of Chandpur (see Figure G.3.4.1 and G.3.4.2).

G.3.4.3 Risk Analysis

G.3.4.3.1 Introduction

For the design of the bank protection a risk analysis has been carried out. The objectives of the Risk Analysis are:

- to define an acceptable probability of failure of the emergency bank protection;
- to identify and quantify the hazards of the emergency bank protection;
- to integrate the design of the protection into other infrastructural works.

In Chapter G.1 of this Annex a more elaborated description is given of the risk analysis.

G.3.4.3.2 Failure modes

An overall fault tree for the advanced bank protection is presented in Chapter G.2. For more information on failure modes reference is made to Section G.1.5.4.2. When preparing this fault tree, the specific conditions at the site of Chandpur have been considered including the Emergency protection. The major difference with the fault tree presented in Chapter G.2 is that in case of emergency protection outflanking is a potential failure mechanism. For the complete protection works which have been selected as Short Term Measures in Chapter G.2 this failure mechanism is negligible.

G.3.4.3.3 Acceptable probability of failure

The acceptable probability of failure of an overall bank protection for Chandpur has been presented in Chapter G.2; the acceptable failure probability arrived at there [illegible]

The design of emergency protection works will imply the re-evaluation of the overall probability failure of the protection because the risk of outflanking has to be taken into consideration as well. This is reflected in the fault tree as shown in Figure G.3.4.3. by adding the failure mechanism of outflanking.

Below, the major failure mechanisms as presented in the aforementioned fault tree will be discussed.

ma

(a) Geotechnical stability

As the slope stability calculations indicate that for the selected slopes of 1:3.5 the safety coefficients are sufficiently high, the failure probability is quite low (see Geotechnical Investigations, Annex C). Nevertheless, for safety reasons, the probability for geotechnical failure is set at 0.125E-03 per year. The latter is equal to the figure as presented in Chapter G.2 because there is no reason why geotechnical conditions are different compared to the proposed short term measures there.

(b) Failure outer slope

The probability of failure of the outer slope is the same as formulated in Chapter G.2 for the short term measures where is was set at 0.125*10-3 per year.

(c) Outflanking

The probability of outflanking is estimated at 1.0*10-1 and is based on Consultants' engineering judgement. This estimate of the failure probability of the outflanking is, inter alia, based on Consultants observations of the outflanking process upstream of the existing emergency protection works at Nutan Bazar carried out in 1990. The same holds for protection works carried out at Puran Bazar. This process is clearly shown in Figures G.3.4.1 and G.3.4.2 for Nutan Bazar and Puran Bazar respectively.

Bearing in mind the period between completion of the aforementioned emergency works (1990) and the present outflanking upstream of the processes occurring here can also be expected upstream of the now proposed emergency works.

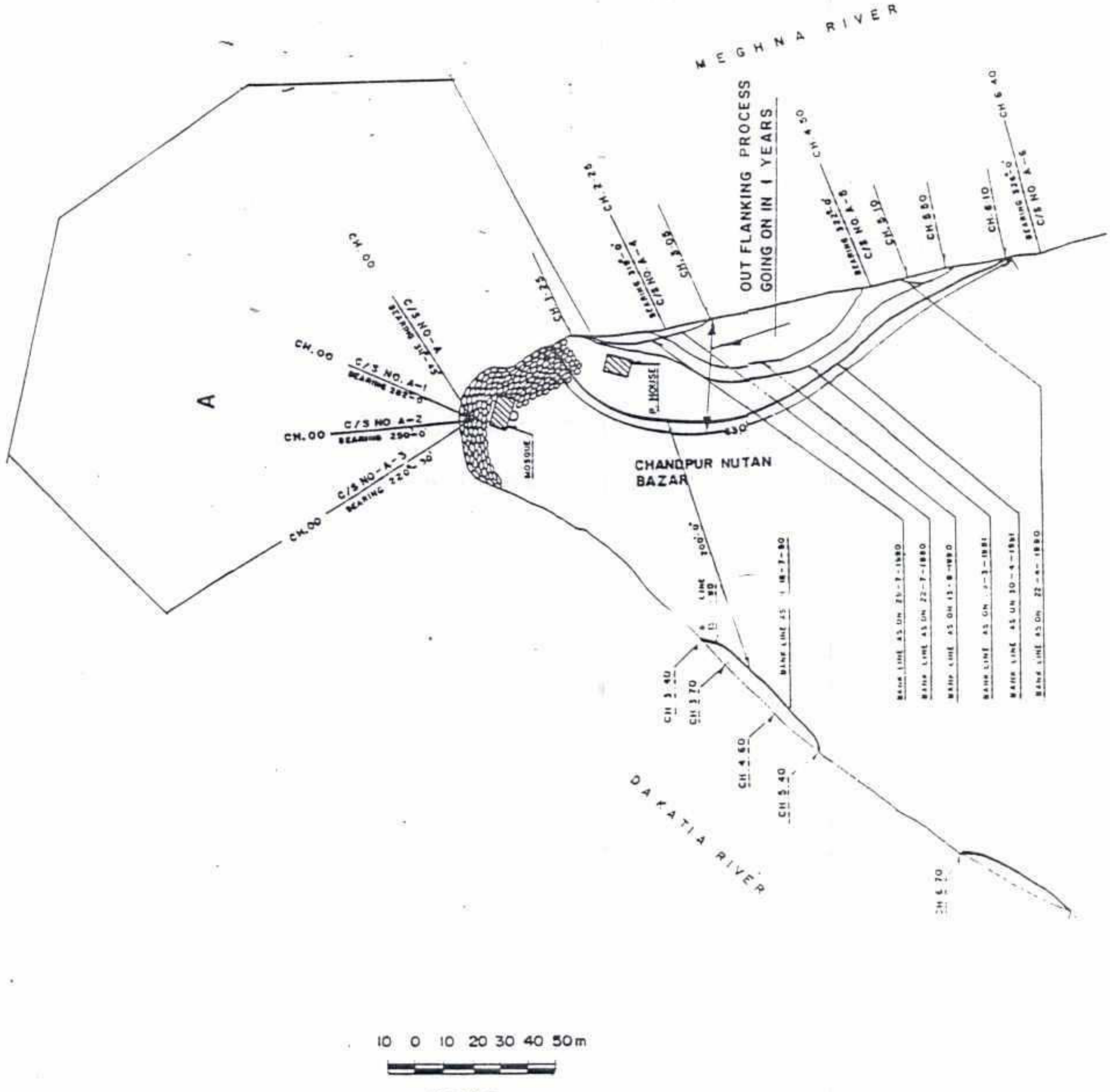
The same process of outflanking can be observed upstream of the emergency works at Puran Bazar.

The probability of 2.0E-01, assigned for failure of monitoring and maintenance is based on engineering judgement.

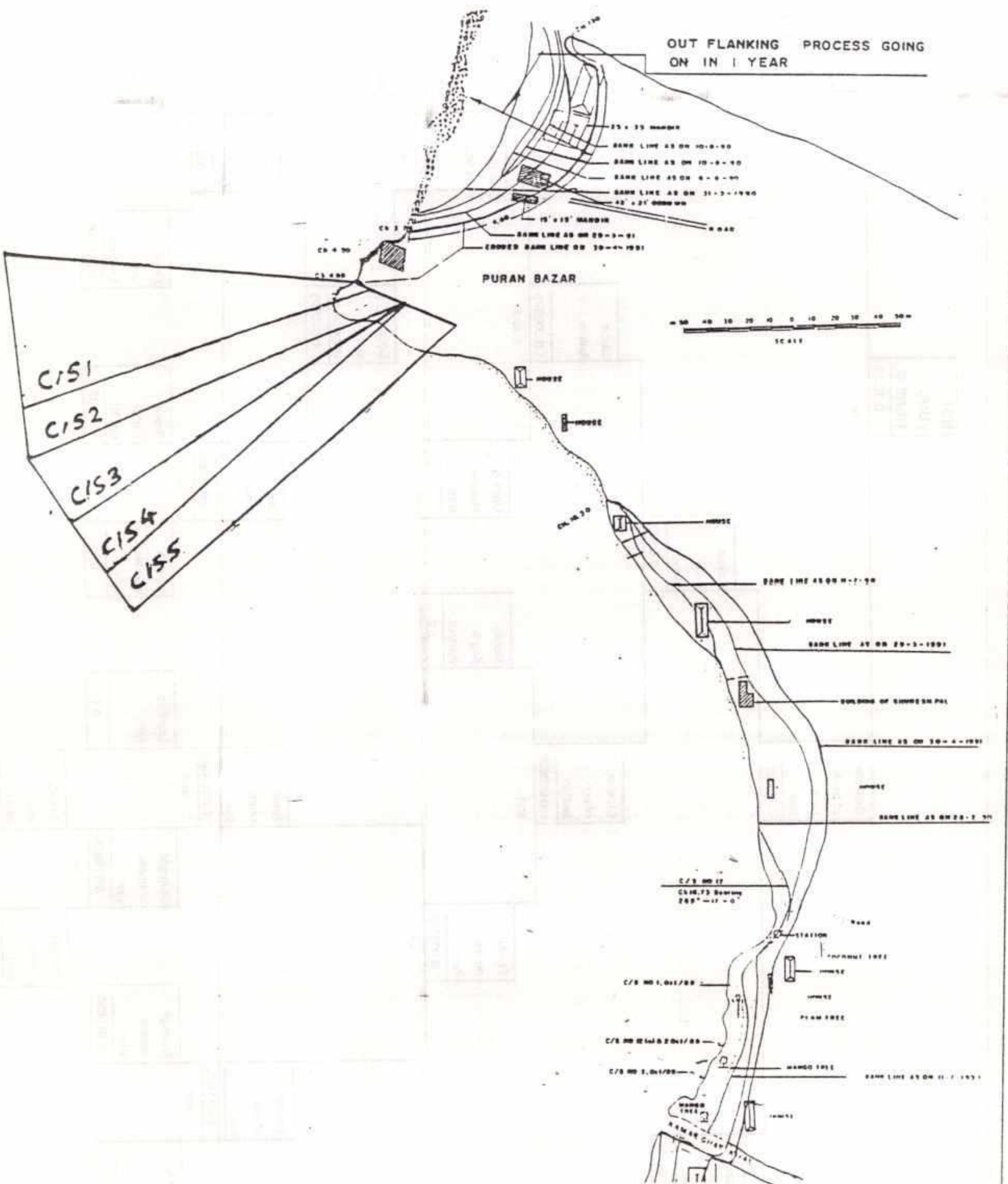
G.3.4.4 Probabilistic calculations

For the results of probabilistic calculations for the outer slope reference is made to Chapter G.2. The latter includes also the probabilistic calculations concerning the scour depths and related dimensions of the falling apron.

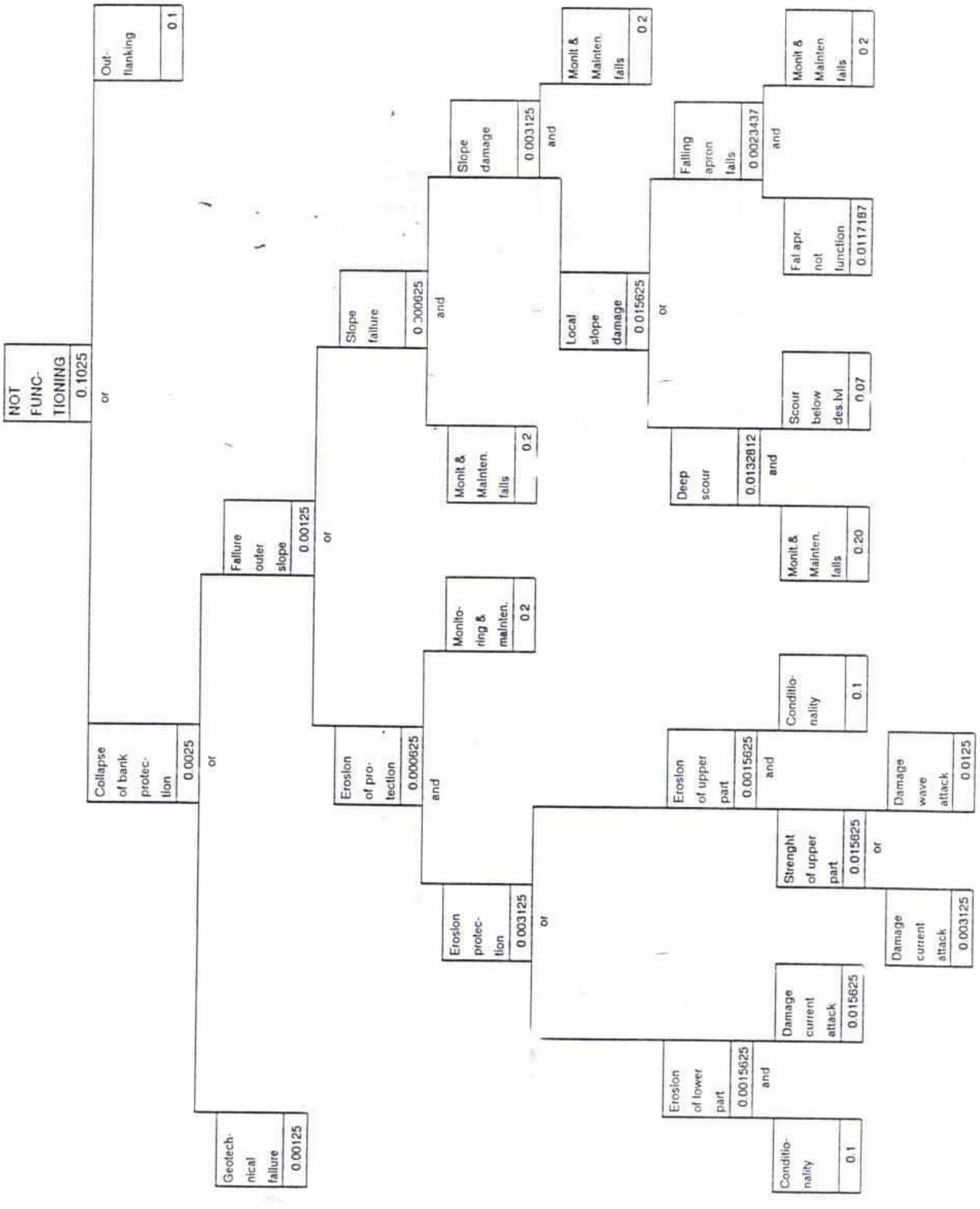
The effects of the risk of outflanking are shown in the fault tree. The overall failure probability is evaluated to be 0.1025 per year. This value is higher than the aforementioned acceptable failure probability for the short-term permanent works. The Consultants are therefore of the opinion that construction of this emergency protection works must be evaluated as a first phase of the short-term protection works at Nutan Bazar. This work will be another temporary work without the commitment to integrate it within the short-term measures; thus, it will not satisfy the requirements, i.e sustainability.



EROSION PROCESS NUTAN BAZAR | FIGURE G.3.4.1



EROSION PROCESS PURAN BAZAR	FIGURE G.3.4.2



FAULT TREE EMERGENCY PROTECTION WORKS
CHANDPUR

FIGURE G.3.4.3

G.3.5 DESIGN

Based on the foregoing Consultants prepared designs for the emergency protection works. In Figure G.3.3.1 a layout is presented whereas in Figure G.3.3.2 a typical cross section is presented.

G.3.6 **CONSTRUCTION METHODS**

As the design concept of the emergency protection works is similar to that of the short term measures reference is made to Chapter G.2, Section G.2.6.

G.3.7 **COST ESTIMATE**

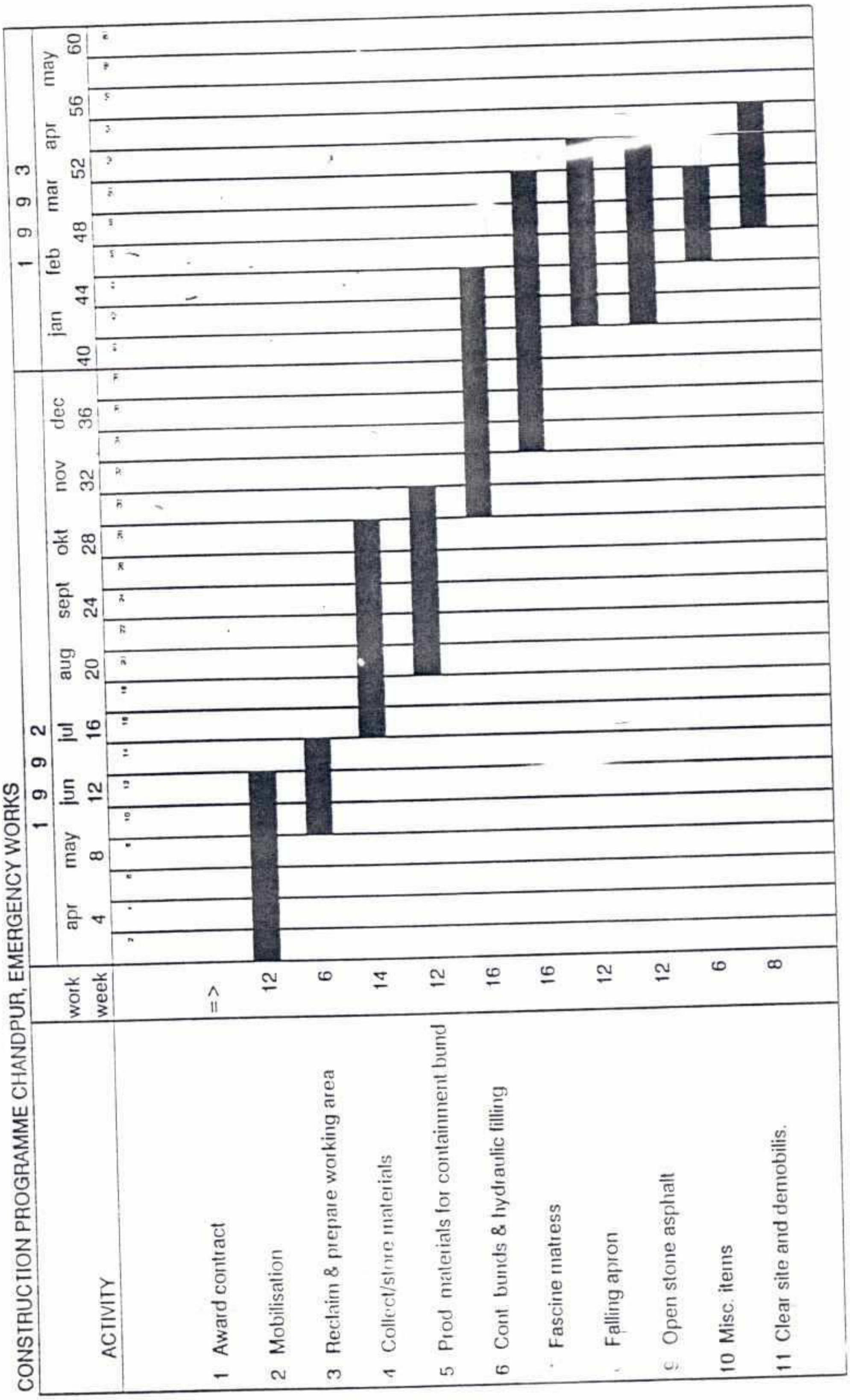
The estimated cost as presented in Table G.3.1 for the construction of the emergency protection works amounts to US$ 37.9 million.

A breakdown is presented in the following Table.

Table G.3.7.1 COST ESTIMATE EMERGENCY PROTECTION WORKS

Item	Quantity	Unit	Unit Cost (US$)	Cost in US$
1 Dredging	2,381,963	m3	3.03	7,228,948
2 Working area/materials	1	-	173,956	173,956
3 Earthworks above SLW	1	-	177,390	177,390
4 Clear site and reinstate	1	-	130,086	130,086
5 Open stone asphalt	12,000	m2	35.12	421,497
6 Fascine Mattress	102,000	m2	48.03	4,898,761
7 Rock in falling apron	37,000	m3	69.73	2,579,887
8 Grouting of rock	821	m	62.58	51,380
9 Containment bunds	210,000	m3	35.45	7,444,600
10 Contractors cost and supervision	1	-	752,000	752,000
11 MOB and DEMOB excl.dredging equipm.	1	-	88,000	88,000
12 MOB and DEMOB dredging equipment	1	-	2,281,578	2,281,578
				=======
				26,228,084
12 Physical contingencies	15	%		
13 Contractors margins and fees	22	%		3,934,213
14 Engineering and supervision	7.5	%		5,770,178
				1,967,106
TOTAL				=======
				37,885,581

A construction program is shown in Figure G.3.7.1



CONSTRUCTION PROGRAM
EMERGENCY PROTECTION WORKS CHANDPUR

FIGURE G.3.7.1

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Appendix G/1

MULTICRITERIA ANALYSIS

APPENDIX G/1 MULTICRITERIA ANALYSIS

G/1.1 Introduction

The selection of different alternatives in relation to the consideration of various criteria, such as structural, social and economic aspects, is difficult to make objectively. In order to obtain a more objective selection, several methods are available:

(i) monetary evaluation methods,
the advantages and disadvantages of the alternatives are expressed in monetary terms;

(ii) non-monetary evaluation methods,
qualitative consideration of various alternatives.

One of the non-monetary evaluation methods, the so called Multi Criteria Analysis (MCA) matrix method, is briefly explained here by means of an example. The outline is extracted from "Guide for Methodology of Selection of Dyke and Bank Protection Works, Part 1, Technical Advisory Committee on Water Protection Works", published in the Netherlands (in Dutch) in 1988.

Ideally a qualitative judgment should be obtained in discussions by a panel of representatives of each discipline involved. The results must be related to the cost calculations of the alternatives. A balanced choice can then be made (best value for money approach).

The method will be clarified by executing the method on an imaginary example with selected criteria A, B and C. In the following example two alternatives, I and II, are evaluated. The criteria considered here can be separated into primary and secondary criteria (see Table G/1.1).

Table G/1.1 PRIMARY AND SECONDARY CRITERIA

primary criteria	secondary criteria
A	1
	2
	3
B	1
	2
C	1

G/1.2 Evaluation of primary criteria

The first stage of the MCA is to determine the relative importance of the primary criteria by means of a so-called norm value. The primary criteria are given in a matrix with horizontal and vertical axes and it is filled in as described hereafter. The criteria are judged by giving a mark in the range of 1 to 3. The significance of the marks is :

1 = row criterion is more important than column criterion
2 = both criteria are equally important
3 = column criterion is more important than row criterion

An example of a matrix with the marks is given in the following Table G/1.2.

Table G/1.2 DETERMINATION OF NORM VALUE FOR PRIMARY CRITERIA

column criteria (primary criteria)	row criteria (secondary criteria) A	B	C	score	Z %
A	-	1	3	4	33
B	3	-	2	5	42
C	1	2	-	3	25
				12	100%

The score of each primary criterion follows by addition of the horizontal axis: the value Z. This score is here called the primary norm value or the weight of the primary criterion.

In this example, the marks range from 1 to 3; it is, however, also possible to take a range from 0 to 10.

G/1.3 Evaluation of secondary criteria

For each primary criterion the secondary criteria must be judged on their relative importance. This can be done by allocating each primary criterion a value of 100 % and assigning a part thereof to the secondary criteria (total should be 100 %). In Table G/1.3 the weight values for the secondary criteria are given by value X.

Table G/1.3 DETERMINING NORM VALUES FOR SECONDARY CRITERIA

primary criteria	secondary criteria	X (%)	
A	1	20	
	2	30	
	3	50	sum = 100%
B	1	70	
	2	30	sum = 100%
C	1	100	sum = 100%

The total of secondary norm values must be 100%. One must realize that the choice of and the number of secondary criteria must be done carefully.

G/1.4 Evaluation of alternatives

For each of the alternatives a mark Y will be given indicating the suitability of the alternative for each criterion. The marks indicate:

Y=0 satisfies requirements almost not at all, to poorly
Y=1 satisfies requirements poorly to sufficiently
Y=2 satisfies requirements sufficiently to reasonably
Y=3 satisfies requirements reasonably to well

The codes for suitability are given in Table G/1.4.

Table G/1.4 CONSIDERATION OF SUITABILITY FOR 2 ALTERNATIVES

primary criteria	secondary criteria	Alternative I	Alternative II
A	1	2	1
	2	3	2
	3	1	3
B	1	2	2
	2	1	1
C	1	3	2

It should be realized that an increase in the number of alternatives makes the exercise more complex and the differences between scores smaller.

G/1.5 Score of the alternatives

Once all the judgments have been given the score for each of the alternatives can be calculated by the following formula :

total score alternative = $\Sigma (\Sigma (X * Y) * Z) * 100$

where:
X = weight of secondary criteria in %
Y = suitability of alternative in points
Z = weight of primary criteria in %

For the above example the results of the calculations are presented in Tables G/1.5 and G/1.6. W is the score of each of the criteria.

Table G/1.5 CALCULATION OF SCORE FOR ALTERNATIVE I

primary criteria	secondary criteria	Z	X	Y	W
A	1	33	20	2	13.2
	2		30	3	29.7
	3		50	1	16.5
B	1	42	70	2	58.8
	2		30	1	12.6
C	1	25	100	3	75.0
					$\Sigma = 205.8$

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Table G/1.6 CALCULATION OF SCORE FOR ALTERNATIVE II

primary criteria	secondary criteria	Z	X	Y	W
A	1	33	20	1	6[illegible]
	2		30	2	19.8
	3		50	3	49.5
B	1	42	70	2	58.8
	2		30	1	5.3
C	1	25	100	2	50.0
					$\Sigma = 189.9$

G/1.6 Evaluation

The scores are summarized in Table G/1.7.

Table G/1.7 SCORE OF ALTERNATIVES I AND II

	alternative I	alternative II
total score	205.8	189.9

The results show that alternative I better satisfies the selection criteria considered.

The final selection should be based not only on the final score in the non-monetary MCA, but also on:

- capital costs,
- maintenance,
- interest,
- period of consideration.

Dividing the final score of the MCA by the total cost, gives an idea of the best "value for money".

Appendix G/2

WEIGHING FACTORS

APPENDIX G/2 WEIGHING FACTORS

In the following Table weigh factors are obtained for selection of the slope protection. For more details reference is made to Appendix G/1.

Table G/2.1 OBTAINING WEIGH FACTORS

	Functional requirements	Quality Assurance	Maintenance	Construc tion	Σ (row)	Weighing
Functional requirements	0	3	3	3	9	0.40
Quality assurance	1	0	2	2	5	0.20
Maintenance	1	1	0	3	5	0.20
Construction	1	2	2	0	5	0.20
					Σ - 5	Σ - 1

The codes indicate:
column > row 3
row > column 1
column = row 2

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Appendix G/3

A THEORETICAL APPROACH TO MAINTENANCE

APPENDIX G/3 A THEORETICAL APPROACH TO MAINTENANCE

Maintenance is important in a long lifetime of the protection works is desired. Moreover, if maintenance is a design parameter, which has been considered during the different design stages, it will decrease the total initial investment. The maintenance cost, however, will increase, but the overall cost, initial and maintenance, during the lifetime of the protection works will be less. Thus, as part of this study, maintenance has been looked at taking these considerations into account.

An approach which can be followed is the following: During the design stage of the project an estimate should be made of the maintenance required for several alternative solutions proposed. A parameter which can describe the level of maintenance is introduced here as the damage parameter. The damage parameter gives an indication as to the condition of the protection works after a certain number of years in terms of area of protection works lost.

The following formula can, amongst others, be used for calculation of the damage factor:

$$p_d = \frac{n_s}{n_0}$$

$$n_0 = \frac{l_s\, b_s}{D_{50}^2}$$

$$n_s = \frac{\Delta t\, q_s\, b_s}{\frac{\pi}{6}\, D_{50}^3}$$

$$q_s = \phi \sqrt{g\, \Delta\, D_{50}^3}$$

$$\phi = 1{,}64\ 10^{10}\ \psi^{11}$$

$$\psi = \frac{\tau}{\rho\, g\, D_{50}}\, \frac{1}{k}$$

where

p_d - *damage level* (-)
D_{50} - *characteristic diameter* (*m*)
l_s - *transportation length* ($\approx 20\ D_{50}$)
q_s - *transport capacity* ($m^2/m'/s$)
Δt - *duration of hydraulic attack* (sec)
b_s - *width considered* (*m*)
k - *slope factor* (-)
n_0 - *original number of stones* (-)
n_s - *number of stones which have been damaged* (-)

The formula can be applied for slope protection works which consist of boulders and rip-rap. Depending on the dimensions of the elements a sensitivity analysis can be made of the maintenance required during the lifetime of the project. Use can be made of probabilistic calculations.

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Appendix G/4

PROBABILISTIC CALCULATIONS

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APPENDIX G/4 PROBABILISTIC CALCULATIONS

G/4.1 **Listing of reliability functions**

G/4.1.1 Stability formula Pilarczyk

```
c*******************************************************
c  PILARCZYK FORMULA FOR STONE STABILITY BHAIRAB BAZAR
c*******************************************************

c*******************************************************
c  DECLARATION OF VARIABLES
c*******************************************************

      Real Discharge , Islope , Rhost
      Real Dn , Angle , Phi , Kstab , Kt , Psicr
      Real Bedlvl , Chezy , Hloc , Rhowat , D50
      Real Stage , Hriver , Veloc , Delta , Kslope
      Real Dum , G , Pi , Kh

c*******************************************************
c  LIST OF SYMBOLS USED
c*******************************************************
c
c  Discharge   = river discharge  (m3/s)
c  Islope      = hydraulic slope (-)
c  Rhost       = density stone (kg/m3)
c  Rhowat      = density water (kg/m3)
c  D50         = 50% value of size distribution of elements (m)
c  Dn          = nominal dimensions element (m)
c  Angle       = slope of protection (degrees)
c  Phi         = angle of friction (degrees)
c  Kstab       = stability factor (-)
c  Kt          = turbulence factor (-)
c  Kh          = influence factor water depth (-)
c  Kslope      = influence factor slope (-)
c  Psicr       = critical shear stress (-)
c  Bedlvl      = minimum river bottom level (m -PWD)
c  Chezy       = Chezy coefficient (m0.5/sec)
c  Hloc        = location in vertical referred to water level (m)
c  Veloc       = mean velocity (m/s)
c  Stage       = water level referred to PWD (m +PWD)
```

Stability formula Pilarczyk (continued)

```
c*****************************************************
c  READ INPUT DATA
c*****************************************************

    Discharge   =      x(1)
    Islope      =      x(2)
    Rhost       =      x(3)
    Rhowat      =      x(4)
    D50         =      x(5)
    Angle       =      x(6)
    Phi         =      x(7)
    Kstab       =      x(8)
    Kt          =      x(9)
    Psicr       =      x(10)
    Bedlvl      =      x(11)
    Chezy       =      x(12)
    Hloc        =      x(13)

    Pi  =    3.141593
    G   =    9.813
    Dn  =    0.85 * D50

c*****************************************************
c   DEFINITION OF RELIABILITY FUNCTION
c*****************************************************

    Stage       =      1.0 + (Discharge/1170.)**0.67
    Hriver      =      Bedlvl + Stage
    Veloc       =      Chezy * ( (Hriver * Islope * 0.00001 )**0.5  )
    Kh          =      (Hloc / Dn ) ** (-0.2)

    If (( Hloc/Dn) .GT. 50.0) Kh = 0.33

c*****************************************************
c  Kh is limited to 0.33 if Hloc/Dn is greater then 50
c*****************************************************

    Kslope      =      (1.-((sin(Angle*Pi/180.)/sin(Phi*Pi/180.))**2))**.5
    Delta       =      (Rhost -Rhowat)/1000.
    Dum         =      (Kh*Kstab*kt*0.035*Veloc*Veloc)/(2*G*Psicr*Kslope)
    Z           =      Dn * Delta - Dum

    End
```

G/4.1.2 Scour calculation Bhairab Bazar

```
c*************************************************
c Scour development Bhairab Bazar
c*************************************************

c*************************************************
c  DECLARATION OF VARIABLES
c*************************************************

      Real Discharge , Hdominant , k1 , k2 , k3
      Real Scourlvl , PrsBedlvl , Waterlvl100
      Real Alpha , Stage , Hinit , Hconscour
      Real ConScour , BdCS , Haverage
      Real Hbendscour , BendScour , BdBS , Horig
      Real LocScour , Z

c*************************************************
c  LIST OF SYMBOLS USED
c*************************************************
c
c     Discharge       =     river discharge (m3/s)
c     Hdominant       =     average water depth during dominant discharge (m)
c     k1              =     model factor (-)
c     k2              =     model factor (-)
c     k3              =     model factor (-)
c     Scourlvl        =     scour depth referred to initial bed level (m)
c     PrsBedlvl       =     presently observed bed level (m -PWD)
c     Waterlvl100     =     Water level during 1:100 flood (m -PWD)
c     Alpha           =     model factor (-)
c     Stage           =     river stage (m +PWD)
c     Hinit           =     initial water depth (m)
c     Hconscour             water depth due to constriction scour (m)
c     ConScour        =     scour depth due to constriction scour (m)
c     BdCS            =     bed level after constriction scour (m -PWD)
c     BdBS            =     bed level after bend scour (m -PWD)
c     Horig           =     original water depth (m)
c     LocScour        =     scour depth due to local scour (m)

c*******************************************************
c  READ INPUT DATA
c*******************************************************

      Discharge       =     x(1)
      Hdominant       =     x(2)
      k1              =     x(3)
      k2              =     x(4)
      k3              =     x(5)
      Scourlvl        =     x(6)
      PrsBedlvl       =     x(7)
      Waterlvl100     =     x(8)
      Alpha           =     x(9)
```

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Scour calculation Bhairab Bazar (continued)

```
c***********************************************************
c   DEFINITION OF RELIABILITY FUNCTION
c***********************************************************

    Stage      =      1.0 + (Discharge/1170.0)**0.67
    Hinit      =      Hdominant + 1.8

c***********************************************************
c   1.8 is increase in waterdepth in unconstricted area
c***********************************************************

    Hconscour =      k1 * k2 * Hinit
    ConScour  =      Hconscour - PrsBedlvl - Waterlvl100 - 2.30

c***********************************************************
c   2.30 m waterlevel rise in constriction area
c***********************************************************

    BdCS             =      PrsBedlvl + ConScour
    Haverage         =      Stage + PrsBedlvl
    Hbendscour       =      k3 * Haverage
    BendScour        =      Hbendscour - Haverage
    BdBS             =      BdCS + BendScour
    Horig            =      BdBS + Waterlvl100
    LocScour         =      alpha * Horig

    Z                =      Scourlvl - Locscour

    end
```

G/4.1.3 Scour calculation Chandpur Town

```
c***************************   *************************
c Scour development Chandpur
c******************************************************

c******************************************************
c Declaration of variables
c******************************************************

    Real Hav , Hbankfull , k1 , k2 , Scourlvl , PrsBedlvl
    Real Waterlvl100 , Hinit , Hbend
    Real Hprot , Totscour , Z

c******************************************************
c List of variables used
c******************************************************

c   Hav            =    average water depth during dominant
c                       discharge (m)
c   Hbankfull      =    water depth bankfull discharge (m)
c   k1             =    model factor (-)
c   k2             =    model factor (-)
c   Scourlvl       =    scour depth referred to initial
c                       bed level (m)
c   PrsBedlvl           presently observed bed level (m -PWD)
c   Waterlvl100    =    Water level during 1:100 flood (m -PWD)
c   Hinit          =    initial water depth (m)
c   Hbend          =    water depth due to bend scour (m)
c   Hprot          =    water depth due to protrusion scour (m)
c   Totscour       =    scour depth referred to initial bed level (m)
c   Stagediff      =    stage difference (m)

c***************************************************************
c Input of data
c***************************************************************

   Hbankfull      =    x(1)
   k1             =    x(2)
   k2             =    x(3)
   Scourlvl       =    x(4)
   PrsBedlvl      =    x(5)
   Waterlvl100    =    x(6)
   Stagediff      =    x(7)
```

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Scour calculation Chandpur Town (continued)

```
c*************************************************************
c  Definition of reliability function
c*************************************************************

      Hav = Hbankfull + Stagediff

      Hbend = k1 * Hav

      Hinit = Hbend

      Hprot = k2 * Hinit

      Totscour = Hprot - PrsBedlvl - Waterlvl100

      Z = Scourlvl - Totscour

      end
```

G/4.1.4 Wave attack open stone asphalt

```
c*************************************************
c STABILITY AGAINST WAVES FOR OPEN STONE ASPHALT
c*************************************************

c*************************************************
c  DECLARATION OF VARIABLES
c*************************************************

    Real Hs  , Coef , D

c*************************************************
c  LIST OF VARIABLES USED
c*************************************************
c
c   Hs          =     significant wave height (m)
c   Coef        =     stability coefficient (-)
c   D           =     thickness of protection (m)

c*****************************************************
c   READ INPUT DATA
c*****************************************************

    Hs          =     x(1)
    coef        =     x(2)
    D           =     x(3)

c*************************************************
c   DEFINITION OF RELIABILITY FUNCTION
c*************************************************

    Z  =      D  - Coef * Hs

    End
```

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G/4.2 Results of probabilistic calculations Bhairab Bazar

Current attack rip rap Bhairab Bazar
Beta = 3.5083
Probability of failure = 2.255432E-04

Name	Type	A	B	mu	si	x	%
Discharge	Gumbel	12971	1814	14534.130	2613.63	14581.300	1
Islope (*10-5)	Normal	0	0	2.000	.200	2.198	8
Rho_s	Normal	0	0	2600.000	100.000	2521.901	5
Rhowa	Normal	0	0	1000.000	100.000	1078.001	5
D50	Normal	0	0	.015	.015	.131	13
Alfa	Normal	0	0	15.950	1.600	16.527	1
Phi	Normal	0	0	35.000	3.500	33.780	1
Stbfa	Normal	0	0	1.000	.100	1.099	8
Kt	Normal	0	0	1.500	.150	1.649	8
Psicr	Normal	0	0	.035	.004	.029	18
Bedvl	Normal	0	0	17.000	1.700	18.281	5
C	Normal	0	0	70.000	7.000	82.908	28
Holc	Normal		0	6.000	.000	6.000	0

Z(x) = 8.863854E-04
Number of iterations = 82

Current attack rip rap Bhairab Bazar
Beta = 3.5123
Probability of failure = 2.221529E-04

Name	Type	A	B	mu	si	x	%
Q	Gumbel	12971.00	1814.00	14430.140	2530.306	14474.440	1
i(*10)	Normal	.00	.00	2.000	.200	2.198	8
Rho_s	Normal	.00	.00	2600.000	100.000	2521.650	5
Rhowa	Normal	.00	.00	1000.000	100.000	1078.251	5
D50	Normal	.00	.00	.150	.015	.130	13
Alfa	Normal	.00	.00	15.950	1.600	16.527	1
Phi	Normal	.00	.00	35.000	3.500	33.781	1
Stbfa	Normal	.00	.00	1.000	.100	1.099	8
Kt	Normal	.00	.00	1.500	.150	1.649	8
Psicr	Normal	.00	.00	.035	.004	.029	18
Bedvl	Normal	.00	.00	17.000	1.700	18.282	5
C	Normal	.00	.00	70.000	7.000	82.911	28
Holc	Normal	.00	.00	25.000	.000	25.000	0

Z(x) = 8.953511E-04
Number of iterations = 78

Current attack rip rap Bhairab Bazar
Beta = 2.6794
Probability of failure = 3.687792E-03

Name	Type	A	B	C	mu	si	x	%
Q	Gumbel	12971.00	1814.00	.00	13181.550	1979.964	13187.290	0
i(*10)	Normal	.00	.00	.00	2.000	.200	2.071	1
Rho_s	Normal	.00	.00	.00	2600.000	100.000	2521.662	0
Rhowa	Normal	.00	.00	.00	1000.000	100.000	1028.299	0
D50	Normal	.00	.00	.00	.150	.015	.143	1
Alfa	Normal	.00	.00	.00	26.750	2.600	30.338	31
Phi	Normal	.00	.00	.00	40.000	4.500	32.123	63
Stbfa	Normal	.00	.00	.00	.750	.075	.776	1
Kt	Normal	.00	.00	.00	1.500	.150	1.553	1
Psicr	Normal	.00	.00	.00	.035	.004	.033	1
Bedvl	Normal	.00	.00	.00	17.000	1.700	17.454	0
C	Normal	.00	.00	.00	70.000	7.000	74.735	2
Holc	Normal	.00	.00	.00	30.000	.000	30.000	0

Z(x) = 1.366513E-03
Number of iterations = 65

HASPROB Probabilistic AFDA calculations
Open stone asphalt waves Bhairab Bazar
Beta = 1.0677
Probability of failure = 1.428348E-01

Name	Type	[illegible]	B	mu	si	x	%
Hs	Gumbel	0.55	0.09	0.275	0.569	0.849	96
Coef	Normal	0.00	0.00	0.167	0.017	0.171	2
D	Normal	0.00	0.00	0.150	0.015	0.146	2

Z(x) = 1.559261E-03
Number of iterations = 19

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HASPROB Probabilistic AFDA calculations
Scour depth calculations Bhairab Bazar

Beta = 1.3967
Probability of failure = 8.124800E-02

Name	Type	A	B	mu	si	x	%
Disch	Gumbel	12971.00	1814.00	12563.790	2021.395	12568.970	0
Hdom	Normal	.00	.00	13.000	.000	13.000	0
K1	Normal	.00	.00	1.700	.170	1.842	36
k2	Normal	.00	.00	1.150	.115	1.246	36
k3	Normal	.00	.00	1.200	.120	1.288	28
Scour	Normal	.00	.00	11.500	.000	11.500	0
Prsbl	Normal	.00	.00	17.000	1.000	17.077	0
Water	Normal	.00	.00	7.790	.500	7.790	0
Alpha	Normal	.00	.00	.300	.000	.300	1

Z(x) = 3.174782E-03
Number of iterations = 101

G/4.2 Results of probabilistic calculations Chandpur

HASPROB Current attack rock Chandpur Town
Beta = 2.6346
Probability of failure = 2.211562E-03

Name	Type	A	B	mu	si	x	%
Q	Gumbel	976580	119550	99387.130	13094.40	99390.340	0
i	Normal	0	0	2.200	0	2.237	0
Rho_s	Normal	0	0	2650.000	.200	2634.397	0
Rhowa	Normal	0	0	1000.000	100.000	1015.584	0
D50	Normal	0	0	.350	100.000	.341	0
Alfa	Normal	0	0	26.750	.035	30.659	31
Phi	Normal	0	0	40.000	2.600	31.592	67
Stbfa	Normal	0	0	.750	4.000	.765	0
Kt	Normal	0	0	1.500	.075	1.530	0
Psicr	Normal	0	0	.035	.150	.034	0
Bedvl	Normal	0	0	24.000	.003	24.416	0
C	Normal	0	0	90.000	2.400	93.500	1
Holc	Normal	0	0	24.000	9.000	24.000	0
					.000		

Z(x) = 2.691376E-03
Number of iterations = 165

HASPROB Current attack rock Chandpur Town
Beta = 4.0978
Probability of failure = 2.086679E-05

Name	Type	A	B	mu	si	x	%
Q	Gumbel	97658.00	11955.00	103103.000	14179.900	103140.700	0
i	Normal	.00	.00	2.200	.200	2.425	8
Rho_s	Normal	.00	.00	2650.000	100.000	2555.622	5
Rhowa	Normal	.00	.00	1000.000	100.000	1094.272	5
D50	Normal	.00	.00	.350	.035	.292	16
Alfa	Normal	.00	.00	15.950	1.500	16.413	1
Phi	Normal	.00	.00	40.000	4.000	38.795	1
Stbfa	Normal	.00	.00	1.000	.100	1.122	9
Kt	Normal	.00	.00	1.500	.150	1.682	9
Psicr	Normal	.00	.00	.035	.003	.031	10
Bedvl	Normal	.00	.00	24.000	2.400	26.555	7
C	Normal	.00	.00	90.000	9.000	110.099	30
Hloc	Normal	.00	.00	24.000	.000	24.000	0

Z(x) = 4.407060E-04
Number of iterations = 238

HASPROB Current attack rock Chandpur Town
Beta = 2.8038
Probability of failure = 2.525304E-03

Name	Type	A	B	mu	si	x	%
Q	Gumbel	97658.000	11955.000	10 827.500	16339.960	106863.200	0
i	Normal	.00	.00	2.200	.200	2.362	8
Rho_s	Normal	.00	.00	2650.000	100.000	2586.840	5
Rhowa	Normal	.00	.00	1000.000	100.000	1063.082	5
D50	Normal	.00	.00	.350	.035	.321	9
Alfa	Normal	.00	.00	15.950	1.500	16.264	1
Phi	Normal	.00	.00	40.000	4.000	39.201	1
Stbfa	Normal	.00	.00	1.000	.100	1.088	10
Kt	Normal	.00	.00	1.500	.150	1.632	10
Psicr	Normal	.00	.00	.035	.003	.032	10
Bedvl	Normal	.00	.00	24.000	2.400	25.829	7
C	Normal	.00	.00	90.000	9.000	104.790	34
Holc	Normal	.00	.00	10.000	.000	10.000	0

Z(x) = 3.311339E-04
Number of iterations = 241

HASPROB Probabilistic AFDA calculations
Open stone asphalt waves Chandpur Town
Beta = 1.0474
Probability of failure = 1.474667E-01

Name	Type	A	B	mu	si	x	%
Hs	Gumbel	0.72	0.11	0.001	1.129	1.137	98
Coef	Normal	0.00	0.00	0.167	0.017	0.170	1
Dn	Normal	0.00	0.00	0.200	0.020	0.196	1
Z(x)		=	2.499892E-03				
Number of iterations		=	18				

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HASPROB Probabilistic AFDA calculations
Scour depth calculations Chandpur Town

Beta = 0.8072
Probability of failure = 2.097753E-01

Name	Type	A	B	mu	si	x	%
Hbankfull	Normal	.00	.00	13.00	1.30	13.525	14
k1	Normal	.00	.00	1.60	.16	1.669	85
k2	Normal	.00	.00	2.70	.27	2.817	85
Scourlevel	Normal	.00	.00	14.50	.00	14.50	00
Presentbedlvl	Normal	.00	.00	50.00	5.0	48.346	84
Waterlvl100	Normal	.00	.00	5.50	.50	5.483	17
Stagediffer	Normal	.00	.00	1.00	.10	1.003	15

Z(x) = 2.348709E-02
Number of iterations = 93

Due to the limited probability of occurrence of this event, viz. 30% (see Geo-morphological Study, Annex B) the annual failure becomes 0.30 * 2.097753E-01 = 0.06.

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Appendix G/5

PROBABILISTIC DESIGN METHOD

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APPENDIX G/5 **PROBABILISTIC DESIGN METHOD**

G/5.1 Approach

G/5.1.1 General

The design of bank protection works can be divided into various stages. These range from interpreting the principal order in terms of environmental, hydraulic, geo-technical and engineering characteristics, to the generation of design concepts, the evaluation of alternative concepts and the exhaustive study of the final design. Also should be taken into consideration the construction of the structure with a controlled feedback to design consideration and criteria.
A bank protection is necessary to safeguard the property of landowners along the River Meghna and to maintain the alignment of the River Meghna. The aim of the Meghna River Bank Protection Short Term Study is to improve the given situation. These activities start with problem formulation in such a format that it is possible to develop alternative solutions.

The problem definition takes into account a definition or description of:

- the functional demands resulting from objectives such as to safeguard the property of landowners along the River Meghna and to maintain the alignment of the River Meghna;
- the boundary conditions set by nature such as waves, water levels, soil conditions and the environment;
- the nature of the criteria on which the conceptual designs will be judged with, the quantification of these criteria, in so far as this is possible.

In the following first the deterministic design will be discussed followed by a discussion of the probabilistic design

G/5.2 Deterministic design of bank protection

As shown in Figure G/5.1 a bank protection consist of various elements which all should be taken into consideration during the design stage. The approach will be clarified by means of an example, thus the following will focus on the stability of the top layer. The approach for the other elements is, however, similar.

The following formula can be used to determine the dimensions of loose materials. Use is made of the Pilarczyk formula for the stability of cover layers under current attack by:

$$Z = \Delta_m \, D_{50} \, 0.847 - \phi \, K_t \frac{0.035}{\Psi_{cr}} \, \frac{K_h}{K_s} \, \frac{\overline{u}^2}{2\,g}$$

A deterministic design approach applies the above formula in such a way that one representative value for the load viz. the water velocity determines the diameter of the stone. This representative value is mostly the highest value which has been measured in say 50 years. The effect of random nature of both load characteristics and strength characteristics is not taken into account. The latter, however, is taken care of in a probabilistic design approach. The manner in which a probabilistic design approach fits into a risk analysis is discussed in the following section.

G/5.3 Risk analysis

The three main elements in a risk analysis are hazard, mechanism and consequences. A risk analysis starts with an inventory of the hazards and mechanisms. A mechanism is defined as the manner in which the structure respond to a hazard (see following Figure)

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PREPARING AN INVENTORY OF HAZARDS

FORMULATING THE FAILURE MECHANISM

CALCULATING THE FAILURE PROBABILITIES

QUALIFYING THE CONSEQUENCES

RISK = PROBABILITY * CONSEQUENCE

A combination of hazards and mechanisms leads, with a particular probability, to failure or collapse of the structure or of its components parts.

Finally, the consequences of failure or collapse must be considered. In the event of failure of bank protection as a whole,the relevant damage characteristics, structural damage and duration of load must be estimated. The probability of failure multiplied by the damage or loss constitutes the risk. For optimal design it is essential to weigh the risk against the cost of constructing a stronger structure.

G/5.4 Probabilistic Design Approach

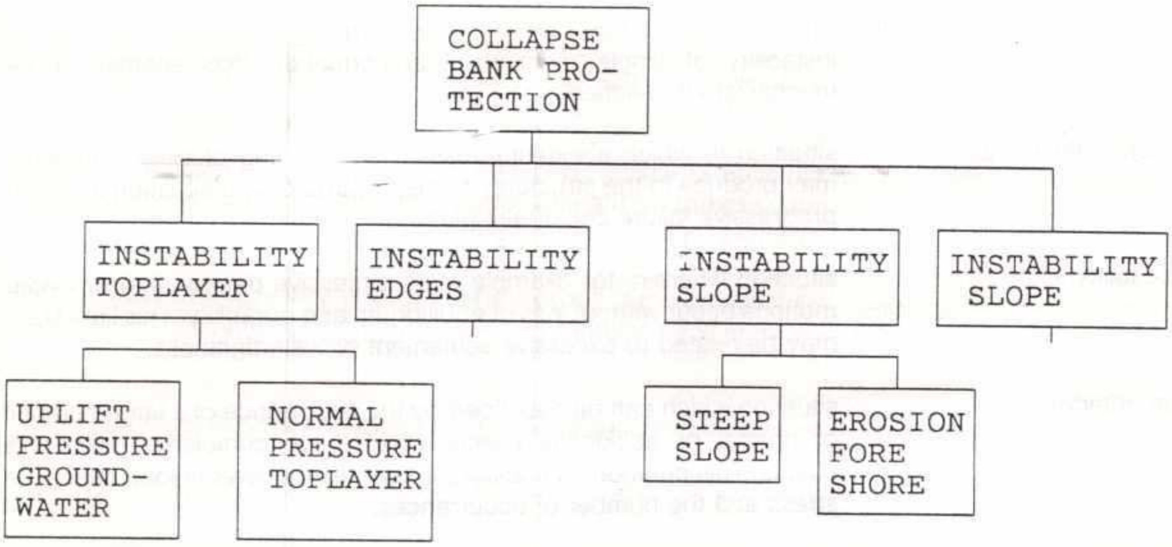
(a) Introduction

The basic concept of a Probabilistic Design Approach is discussed in a global way in the following section. After the description of the concept the successive stages of such a concept will be elaborated upon at a more detailed level.

(b) The Concept

Bank protection works along the Meghna River are constructed to protect the population and the economic values against floods and the alignment of the River. The absolute safety is nearly impossible to realize. Therefore it is much better to speak about the probability of failure of a certain protection system. To apply this method all possible causes of failure have to be analyzed and consequences determined. The so called fault tree is a good tool for this aim. In the following Figure a fault tree for a bank protection is presented.

The fault tree is an essential part of the Probabilistic Design Approach which, as a rule, can only be applied quantitatively at the design stage. The fault tree is a scheme in which events and their consequences, or errors and their causes, which contribute to the failure, are arranged clearly.



Fault tree for bank protection

All possible modes of failure of elements can eventually lead to the failure of a bank protection.

Although all categories of events, that may cause the collapse of a bank protection, are equally important for the overall safety, the engineers' responsibility is mainly limited to technical and structural aspects. In case of a bank protection the following main events can be distinguished :

- erosion of the outer slope or loss of stability of the bank protection;
- instability of the inner slope leading to progressive failure;
- instability of foundation and internal erosion e.g. piping;
- instability of the whole soil body.

For all these modes of failure, the situation where the forces acting are just balanced by the strength of the structure is considered. This is also called the ultimate limit state. The latter implies that the probability density function of possible threat (or loads) and the resistance against it are combined. This will be discussed in more detail further on.

Potential threats are extreme velocities and water level changes. The resistance of a structure is obtained from the basic variables by means of models, both theoretical and empirical.

(c) <u>Limit states</u>

The limit state of a structure is defined as the situation wherein it can just fulfil its functions. A definition of a limit state is "that state at which a structure or structural member reaches a limit of fitness in a condition where it just about ceases to fulfil the requirements of resistance or other specifications related to the structural performance for which it has been designed".

This almost malfunctioning of the structure can be described when the possible failure mode of the structure, or part of it and the load effects are known. As already mentioned these are presented in a fault tree.

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Some limit states are presented in the following:

ultimate limit state — situation where the ultimate load carrying capacity, collapse or instability of single elements, transformation into another failure mechanism is reached.

progressive limit state — situation in which accidental loss or overloading of single elements may produce in the structure, or major parts of it, a situation in which progressive failure could take place.

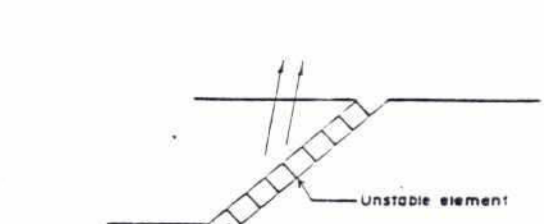
serviceability limit — situation wherein, for example state excessive deformation or cyclic motions occur without loss of equilibrium and durability. This limit state may be related to excessive settlement or watertightness.

fatigue criterion — situation which can be described by the occurrence of a large number of normal or accidental events which have cumulative damaging effects. This criterion, in addition, integrates the level of load effects or stress and the number of occurrences.

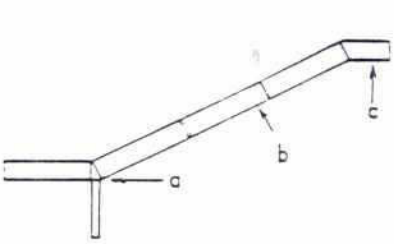
safety domain — this is an area defined by a set of limit states and expressed as a relationship between loads or load effects and strengths of the structure or structure elements.

Some examples of limit states of failure mechanisms are shown in the figures in the following pages:

INSTABILITY OF AN ELEMENT OF THE TOP LAYER



INSTABILITY OF ELEMENTS IN TRANSITIONS



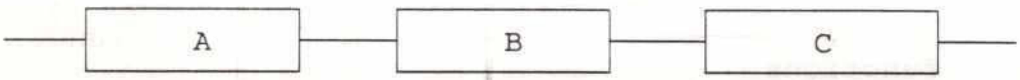
Limit states of failure modes

(d) Fault trees

When assessing the safety of a bank protection it is very important to consider the system as a whole. The structure is composed of many components such as top layer, filter layer and toe structure. Each of these may be prone to many hazards and mechanisms.

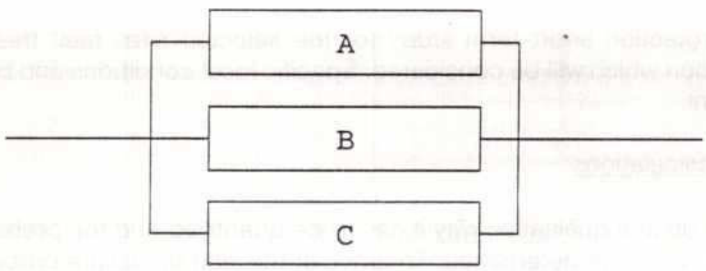
The relation in a fault tree can be considered in two ways viz. a relation between events as a series system and relations like a parallel system.

Collapse of one of the components may in turn pose a hazard to another component. The failure of some components may lead directly to failure of the system. This is called a series system and is presented in the following figure.



Series system

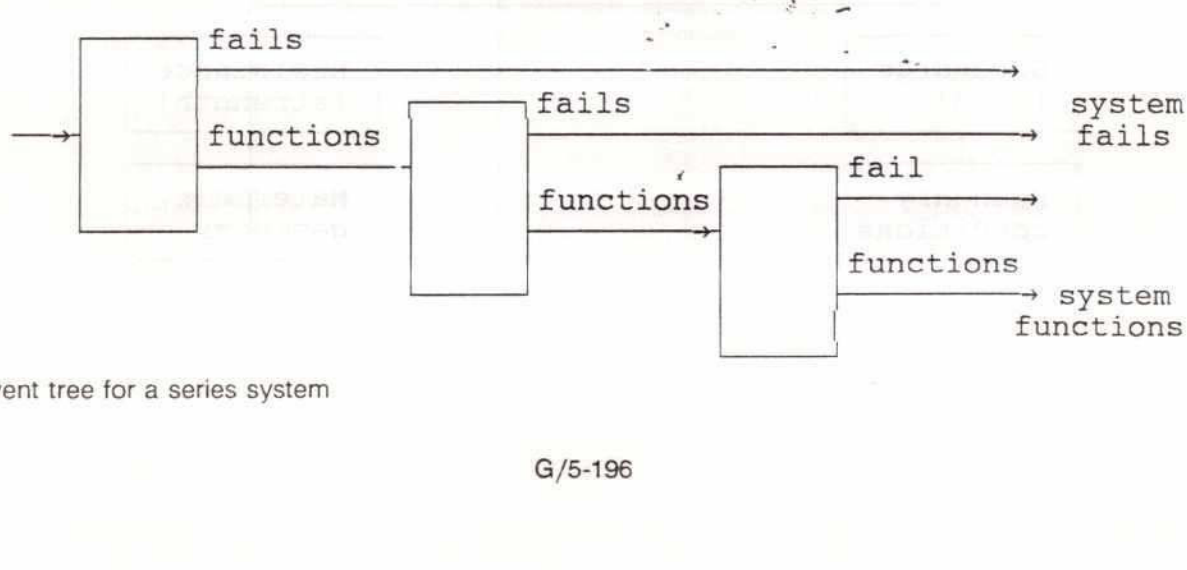
In other cases components may compensate for one another. This is called a parallel relation and is presented in following figure.



Parallel system

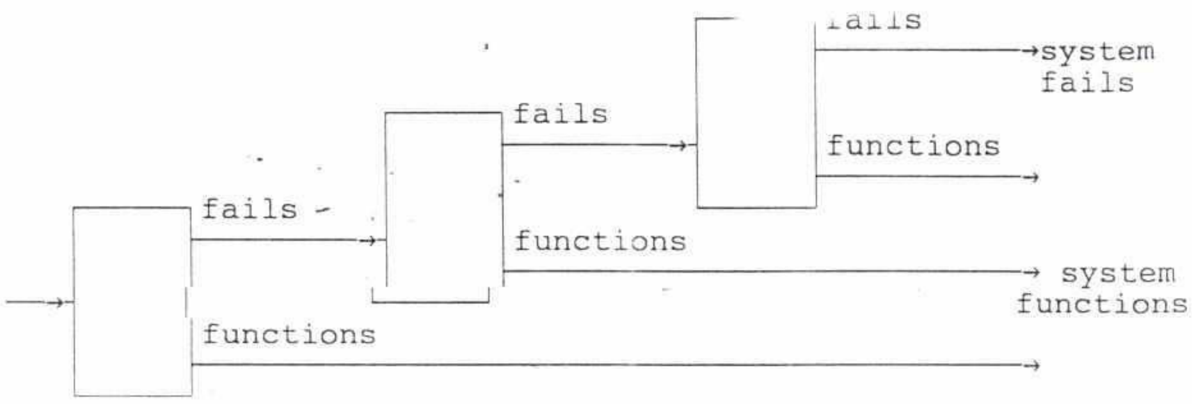
A systematic analysis of the failure behaviour of a structure can be carried out by means of a fault tree analysis. As already mentioned the fault tree presents the possible failure mechanisms and their relation to the so called undesired top event.

To be able to analyze the system with help of the fault tree technique all the possible events of the structure should be separated into two types: failure or functioning. In following in the figure an example of such is given for a series system.



Event tree for a series system

The probability of each of the events and subsequently the probability of the undesired top event can be determined by calculation or estimation. The probability of failure (system fails) or functioning can now be calculated or collected from data collection. If no data are available sound engineering judgement should be applied. For a parallel system a similar event tree can be set up. The following figure gives an example of such a parallel system.



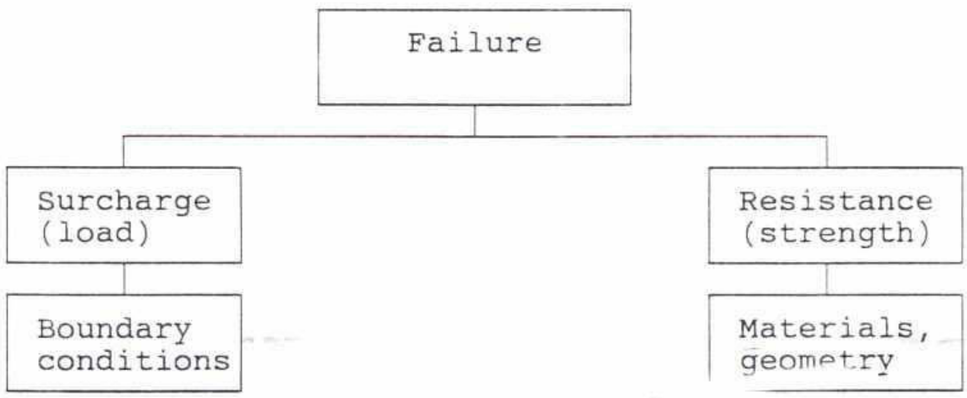
Event tree for a parallel system

In the Meghna River bank protection short term study for the selected sites fault trees have been developed for the bank protection which will be considered. Specific local conditions and circumstances can then be taken into account.

(e) Probabilistic calculations

Once a fault tree has been set up in a qualitative way it has to be quantified and the probability that the undesired top event will occur has to be determined. To arrive at this end the failure probabilities of the different events should be known. These probabilities can be obtained by engineering judgement or by means of probabilistic calculations. In the following the approach for probabilistic calculations in a probabilistic model will be explained.

A probabilistic model describes the loads and strengths which should be considered in each of the events of a fault tree. In the following figure such is model is presented.



Probabilistic model

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The parameters related to load and strength have a random nature, which means that their exact magnitude is not known with certainty. The probabilistic approach therefore expresses this uncertainty in the probability of failure of the structure. The deterministic approach in contrast, gives a safety factor. The probabilistic approach defines however a reliability function Z, where:

$$Z = R - S$$

where:

Z	=	Reliability function
R	=	Resistance or strength
S	=	Surcharge or load

Failure occurs if $Z < 0$. R and S are expressed in statistical terms for one or more basic variables. The relation between R and S is visualized in the previous figure.

The intensity of the loading S is related to the probability of occurrence of the selected initiating event which can be obtained from historic observations and extrapolation techniques. For example the maximum current occurring during a certain time span may follow a Weibull distribution. The resistance R is related to the selected failure mode is to be expressed in a mathematical model form using physical concepts; the fundamental parameters in these formulations are usually stochastic. Their probability distributions are usually considered to be normal or log-normal, and the variables are treated as independent. These assumptions simplify the calculations significantly, but the validity has to be checked.

The limit state of the component considered occurs at $Z=0$; the failure state is related to $Z < 0$. The probability of failure is therefore equal to the joint probability of parameter combinations which correspond to $Z<0$. Mathematically this implies integration of the probability density function of the parameters involved over the domain of failure. Several methods with a different level of sophistication exits by which the calculations are performed. For classifying these techniques the following levels can be distinguished:

level I	Semi-deterministic approach which present constructural design methods where relevant partial factors are used.
level II	Comprises a number of approximate methods in which the problem is linearized. Three well known methods are: first order mean value approach, first order design-point approach and approximate full distributions.
level III	Full-probabilistic approach takes into account the exact joint probability distribution functions including correlations between the parameters.

The advantages of the level II calculations compared with the level III calculations are the gained insight in relative importance of the contributing components on the failure probability. The lesser accuracy of the results of level II calculations if compared to level III calculations is good for engineering purposes. Level II, first order design point approach probabilistic calculations have been performed in the case studies. In the literature this method is frequently referred to as: Advanced Full Distribution Approach (AFDA). The latter has been applied for the design of the bank protection works in the Meghna River.

G/5.5 Risk Analysis and Costs Estimate Bhairab Bazar Railway Bridge

G/5.5.1 Introduction

In order to arrive at damage-frequency curves for Bhairab Bazar Railway Bridge, amongst others, probabilities of occurrence of certain damage cases are required. To determine these probabilities use is made of a risk analysis. This risk analysis is similar to the one applied in Annex G, Chapter G.1. Bank Protection Upper Meghna Sites for the design of the protection works at Bhairab Bazar. Fault trees which have been defined there are also applied in this section, the same holds for the probabilities of failure.

The situation with the proposed protection works and the situation without proposed works is considered.

G/5.5.2 Definition of cases

For purposes of this analysis the Railway Bridge has been schematized into the following basic sub-divisions (see Figure G/5.4.1):

- bridge abutment;
- span 1;
- pile 1;
- span 2.

The failure scenarios may be caused by natural actions and man made actions, or various combinations of these actions. Some natural actions are dead loads, river current and wind loads whereas some man made actions are imposed loads, live loads and train derailment. For purposes of this exercise only sub-divisions which can be negatively effected due to the failure of the bank protection works have been selected.

The following cases have been defined for the risk analysis.

a repairs of the bridge abutment;

b failure of the bridge abutment;

c failure of the bridge abutment and span 1;

d failure of the bridge abutment and span 1 and pile 1;

e failure of the bridge abutment and span 1 and pile 1 and span 2.

The probabilities of failure of the separate events are determined in the following Section.

G/5.5.3 Probability of separate events

In Figure G/5.4.2 and G/5.4.3 fault trees are presented for the 'with' and 'without' situation respectively. The fault tree for the situation with the proposed protection works has already been discussed in Chapter G.1. The failure probabilities in the other fault tree have been derived from of the firstly presented by re-calculating the failure probabilities for the various basic events for the situation without protection.

The failure probabilities of the various events which can be distinguished in the aforementioned cases are discussed below.

i) repairs on bridge abutment

In the fault tree failure of monitoring and maintenance has been assigned at 2.0*10-1 per annum which implies that the probability repairs will occur is 1.0 - 2.0*10-1 = 8.0*10-1.

ii) failure of bridge abutment

It is assumed that the collapse of the bank protection, viz. the top event of the aforementioned fault trees, implies that also the bridge abutment will fail. Therefore the failure probability has been set for the 'with' situation at 5.0*10-1 (see Figure G/5.4.2) and at 1.194*10-1 for the 'without' situation (see Figure G/5.4.3). Taking into account the conditionality (=0.5) the latter values become approximately 5.0*10-2.

iii) failure of span 1

Failure of span 1 can be caused by failure of pile 1 or by sliding away of the bridge abutment. For both it is assumed that failure is mainly caused by failure of the falling apron section initiating a slide of the bridge abutment or collapse of the foundation of the pile. In the fault tree the corresponding values of the slope have been applied; 1.25*10-3 for the 'with' situation and 1.44*10-2 for the 'without' situation.

Failure of span 1 is the result of a 'OR' relation between failure of bridge abutment and failure of pile 1, implying an addition of mentioned values and thus resulting in 2.5*10-3 and 2.88*10-2 respectively.

iv) failure of pile 1

See the previous item [illegible] failure probabilities of pile 1, thus; 1.25*10-3 for the 'with' situation and 1.44*10-2 for the 'without' situation.

v) failure of span 2

The failure of span 2 can be caused due to failure of pile 1. Pile 2, however, is not considered because it assumed not to be part of the affected area of the protection. The failure probabilities are 1.25*10-3 and 1.44*10-2 respectively.

G/5.5.4 Probability of defined cases

The failure probabilities of the defined cases are the result of 'AND' relations between the events, as shown in the fault trees. According to the rules of probabilistics, this means that resulting failure probabilities can be obtained by multiplying the aforementioned failure probabilities of the events.

In the Table G/5.4.1 the results of this exercise are presented.

G/5.5.5 Repair costs

To estimate the repair costs associated with the different failure cases, Consultants used the detailed cost estimate made for the [illegible] bridge in 1990. Assuming that the repairs of the Bhairab Bazar Bridge will be similarly designed some corrections have been applied on the costs of the different elements. These corrections consist of (i) a reduction of 75% of the costs of abutments, pile caps and spans, due to the smaller dimensions (ii) the same costs for a bridge pier in view of the fact that just one pier has to be built and mobilisation/demobilisation costs of pile driving equipment will increase the costs considerably, (iii) an increased percentage (100%) for contractual costs, mobilization and demobilization, engineering and supervision in view of the smaller scope of the repair project, (iv) a lump sum

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for clearance of the failed pier and (v) a 10% upgrade of t[illegible] costs to bring them to the mid-1991 price level. Table G/5.5.2 provides the details.

In this way the repair costs of the failure situations analyzed are summarized as follows:

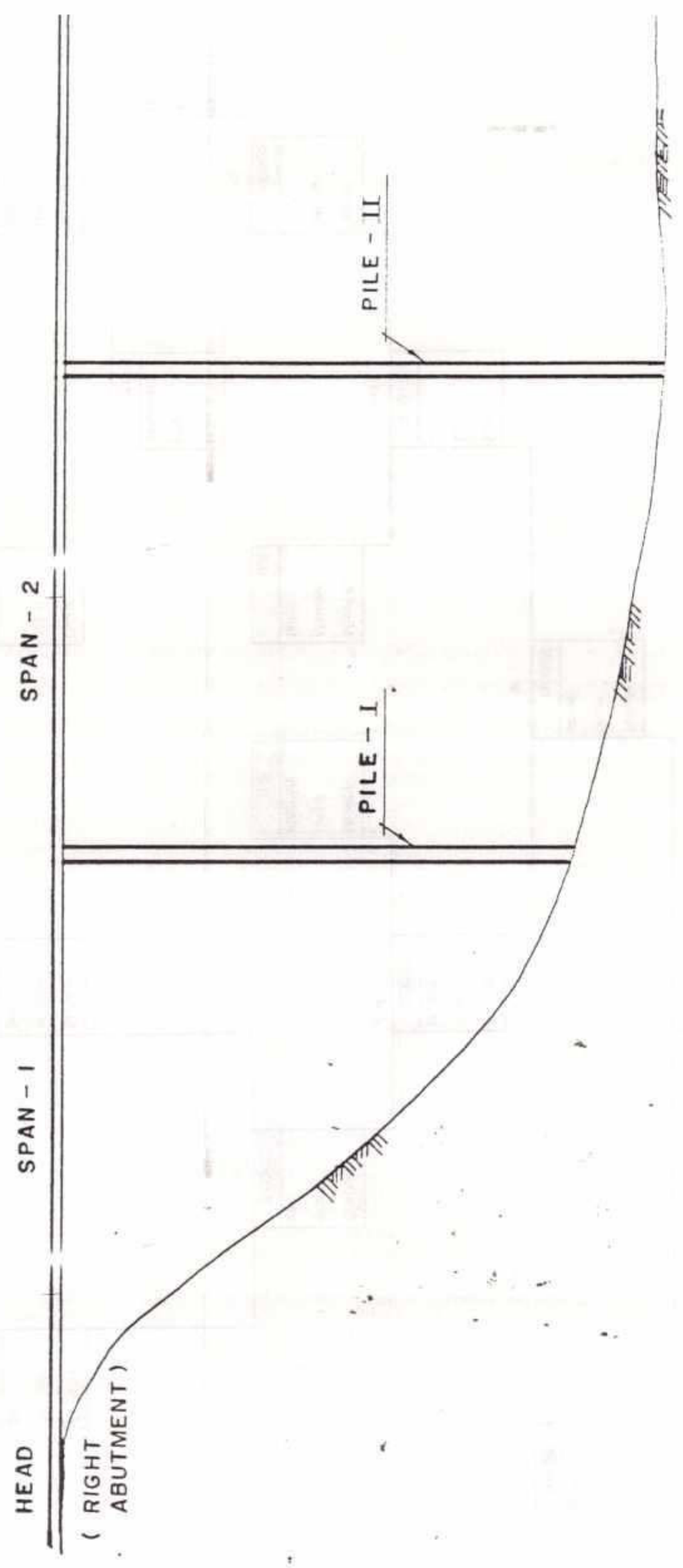
- failure of bridge abutment US$ 2 million
- failure of bridge abutment and span US$ 4 million
- failure of bridge abutment, span and pillar US$ 10 million
- failure of bridge abutment, two spans and pillar US$ 12 million

Table G/5.5.1 PROBABILITIES OF FAILURES FOR 'WITH' AND 'WITHOUT'

Case	Contributing events	Failure probability 'with' protection	Failure probability 'without' protection
a	Pf(I)	8.0*10-1	8.0*10-1
b	Pf(II)	5.0*10-3	[illegible] 10-2
c	Pf(II + III)	1.25*10-5	1.72*10-3
d	Pf(II + III + IV)	6.25*10-8	2.48*10-5
e	Pf(II + III + IV + V)	1.95*10-11	3.57*10-7

Table G/5.5.2 REPAIR COSTS OF BRIDGE ELEMENTS

Element	Jamuna Bridge estimate US$ x 1000	Bhairab Bazar estimate US$ x 1000	Mob/demob, Engineering and Supervision US$ x 1000	Total costs mid-1990 price level US$ x 1000	Total costs mid-1991 price level US$ x 1000	Clearance pier failed US$ x 1000	Total costs US$ x 1000
abutment	800	600	600	1,200	1,320		1,320
cap structure	400	300	300	600	660		660
span	800	600	600	1,200	1,320		1,320
pier	2,200	2,200	2,200	4,400	4,840	1,200	6,000



ELEMENTS OF BRIDGE (INDICATIVE ONLY)	FIGURE G/5.4.1

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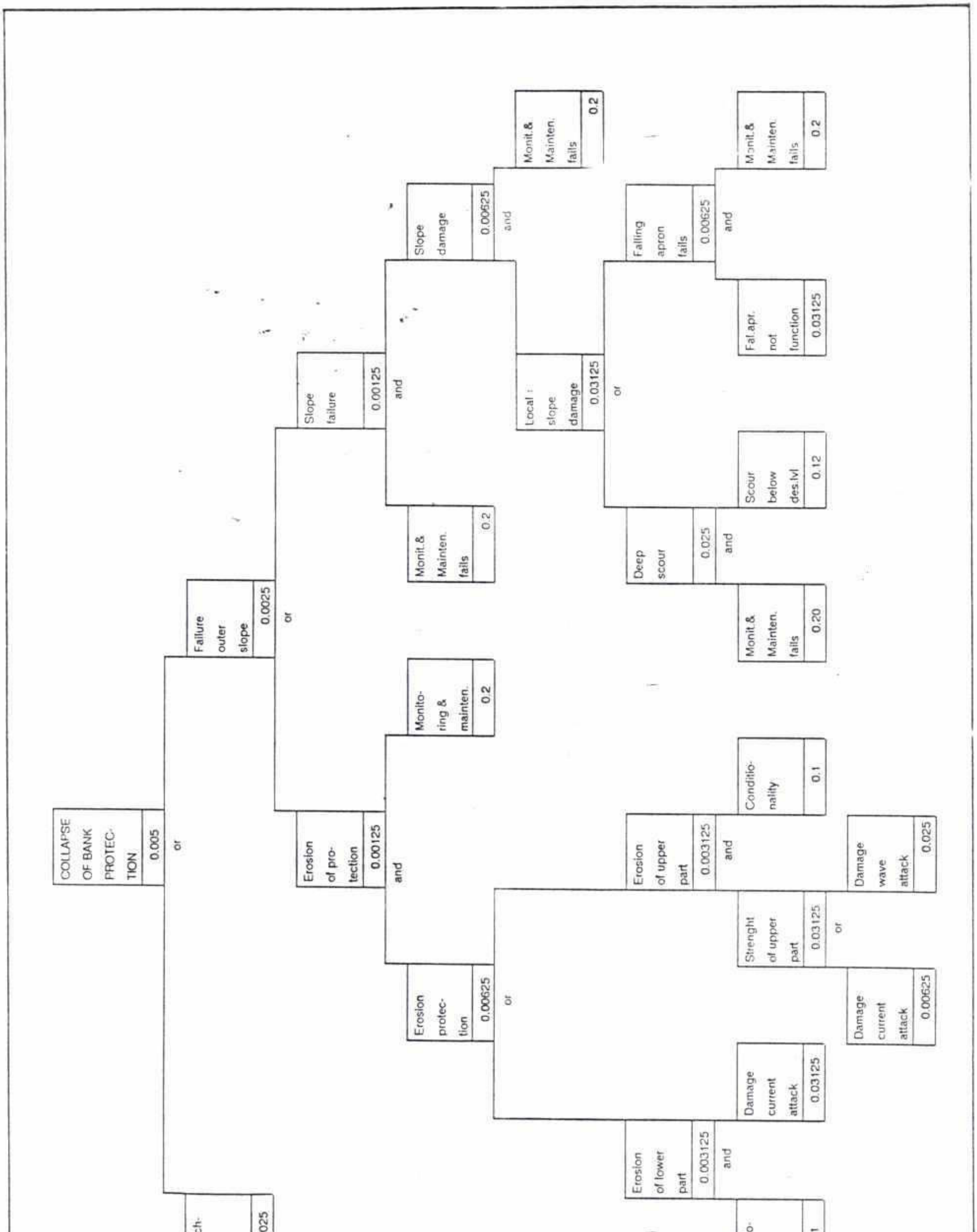
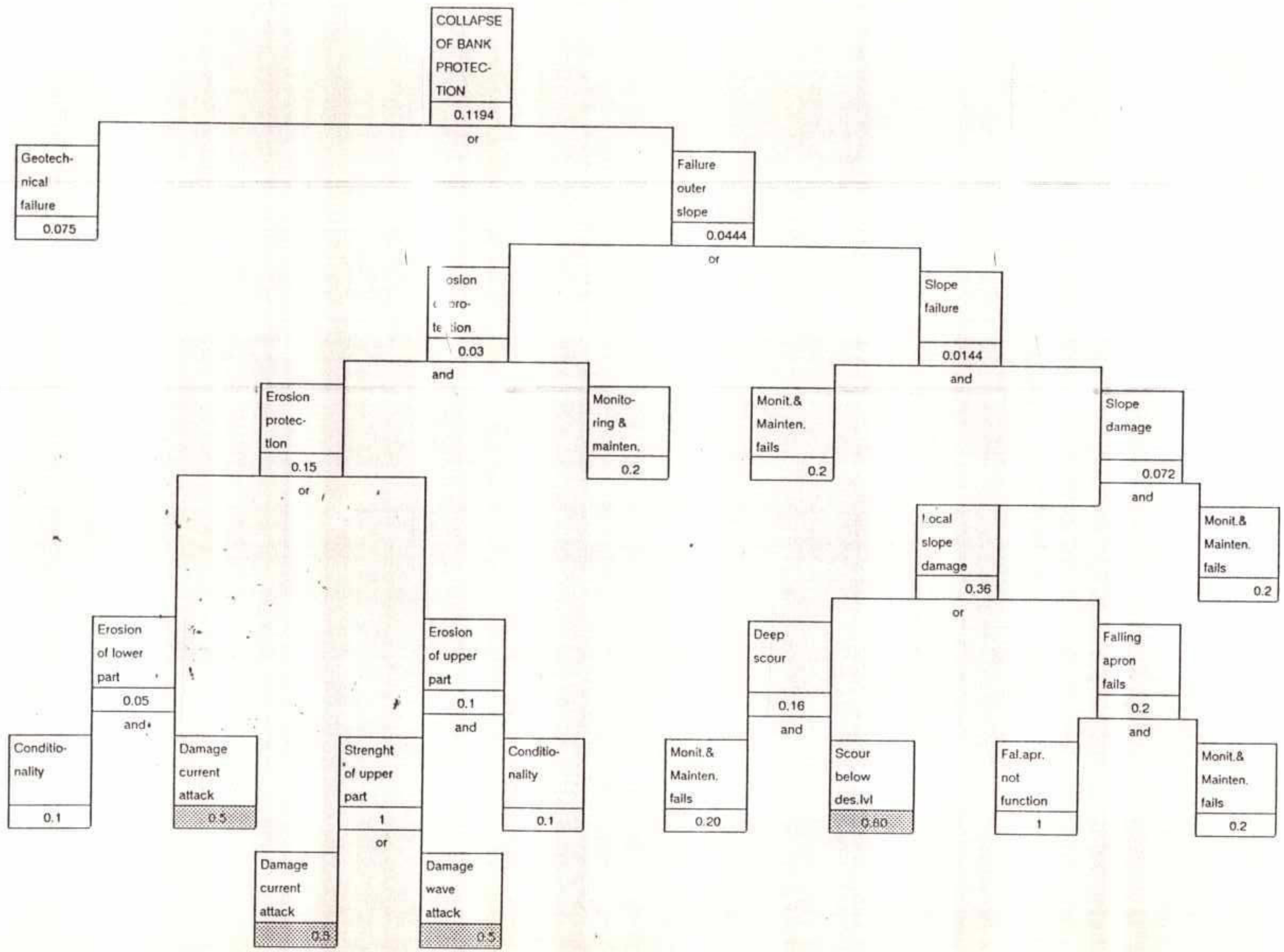


FAULT TREE BHAIRAB BAZAR BRIDGE WITH PROTECTION

FIGURE G/5.4.2



FAULT TREE BHAIRAB BAZAR BRIDGE WITHOUT PROTECTION

FIGURE G/ 5. 4. 3