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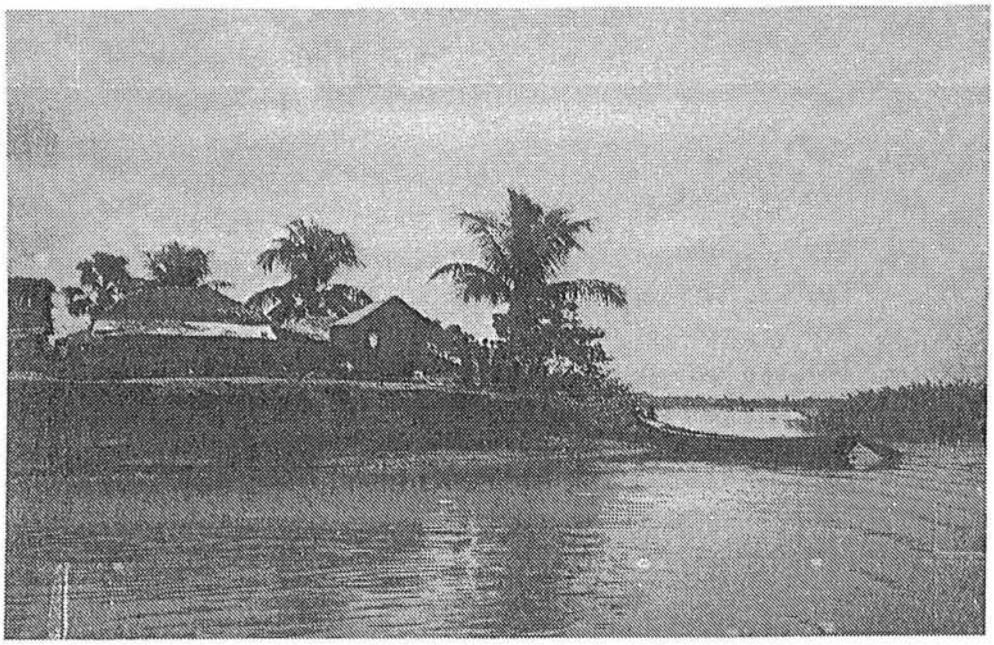

Final report of this thesis study by:

M.F. Guinée.

August 1993

Delft University of Technology
Faculty of Civil Engineering
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in association with

Haskoning
Royal Dutch Consulting
Engineers and Architects
Nijmegen, the Netherlands

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Cover photo: Phenomena of bank erosion at the left bank $\pm$ 1 km. upstream from the Meghna Bridge (september 1992)

PREFACE

This MSc-thesis is the result of a study performed between December 1992 and June 1993. Subject of the study is the location of the "Meghna Roads and Highways Bridge" at the Upper Meghna River in Bangladesh. The study itself is performed at Haskoning, Royal Dutch Engineers and Architects, in Nijmegen and the data collection took place in Dhaka, Bangladesh during September 1992.

For the opportunity to visit Bangladesh and collect all data needed for this study I would like to thank the "Lamminga fonds" from the Technical University of Delft which provided me the travel expenses and Haskoning, which provided me accomodation and transport in Bangladesh.

Furthermore I would like to thank Ir. R.M. Noppeney (Haskoning) who received me in Bangladesh and helped me starting the data collection and the set up of the study. The other persons who helped me in Bangladesh and to whom I would like to express my personal thanks are:

- Mr. Abdul Talha (Roads and Highways Department of the government of Bangladesh)
- Mr. N. Ogawa (Pacific Consultants International[1])
- Mr. T. Kakoto (Obayashi corporation[2])
- Mr. O. Hideya (Obayashi corporation)

For his kind assistance at the Haskoning Offices in Nijmegen I especially would like to thank Mr. F. Carvajal Monar (MSc).

I also would like to thank my supervisors, Ir. J. van Duivendijk, Ir. B. Te Slaa, Prof. Ir. K. d'Angremond and Ir. K.G. Bezuyen for their advice and encouragements during the entire study.

Finally I would like to thank all persons, not mentioned personally above, who were involved in making this study a grant finish to my studies at the Delft University of Technology.

Mark Guinée
Delft, August 1993

[1] P.C.I. is the (Japanese) consultant for the Meghna Bridge.

[2] Obayashi corporation is the (Japanese) contractor for the Meghna Bridge.

ABSTRACT

In 1990, in Bangladesh, the "Meghna Roads and Highways Bridge" was finished. This Bridge is the first one in a series of two Bridges replacing the ferries over the Upper Meghna River. During construction, and also after the finalization of the Bridge, it was revealed that the designers of this Bridge underestimated the power of the Upper Meghna River here.

This revealed itself in various aspects. The most severe ones are the slope failure just upstream from the Bridge which occurred already during the construction, the collapse of (parts of) the slope protection of one of the Bridge's abutments (eight months after the finalization) and the present danger induced by the morphological planform changes of the river just upstream of the Bridge. In the future this could lead to outflanking of the Bridge by the River.

This study discusses the various possible measures which can be taken to safeguard the Meghna Bridge on the long term. Firstly, in part I of this report, the present situation at the location of the Meghna Bridge is described and analyzed. Subsequently, in part II of this Report, a design study is conducted. Designs, solving the present problems, are elaborated onto a detailed design level. These designs are all elaborated with the specific characteristics of Bangladesh being a developing country in mind. Therefore an important aspect during the design study is the use of local resources (both technical as labour) as much as possible.

The design study resulted in three final alternatives. These alternatives are evaluated by means of a Multi Criteria Analysis for their non-monetary criteria. Taking also into account the costs estimates for all alternatives resulted in the recommendation of "alternative III".

This "alternative III" consists of the following. A series of six groynes is foreseen in the river bend just upstream of the Bridge and a guide bank structure replaces the damaged abutment of the Bridge.

A remarkable aspect of this finally recommended design is the fact that all projected structures are to be constructed with a core of sand cement stone and concrete blocks as slope protection. Instead of the (almost classical) method of a soil core with a slope protection of fascine mattresses etc., which is also elaborated being one of the alternatives, it is concluded that the application of sand cement stone as construction material here is the best option.

This is explained by a combination of reasons. As the country of Bangladesh is located in one of the largest deltas in the world its soil consists mainly of the fine alluvial deposits as (rather fine) sand. Therefore rock and boulders are rather scarce construction materials here. Another important aspect is the fact that Bangladesh is a developing country. Therefore "high tech" construction techniques as dredging in deep water and the placing of elements under water are locally not (yet) widely known and available.

Using sand cement stone and concrete blocks as construction materials requires only simple construction techniques. Another merit is the fact that in the structures rather steep slopes can be applied which results in a (relative) saving of material and therefore results in low costs.

These (and others) reasons finally resulted in the recommendation of the mentioned alternative. Drawings of the structures, foreseen within this alternative, are given in the Figures II.21 to II.23.

CONTENTS

ANNEXES:

Abbreviations used:

B.C.L.	Bangladesh Consultants Limited.
B.E.T.S.	Bangladesh Engineering and Technological Services Ltd.
B.I.W.T.A.	Bangladesh Inland Water Transport Authority.
B.W.D.B.	Bangladesh Water Development Board.
C.P.T.	Cone Penetration Test.
F.A.P.	Flood Action Plan.
J.I.C.A.	Japan International Cooperation Agency.
kg	kilograms.
km	kilometres.
km^2	square kilometres.
kN	kilonewton.
L.W.L.	Low Water Level.
m.	meter(s)
M.C.E.	Multi Criteria Evaluation.
m/s.	meters per second.
m^2	square meters.
m^3	cubic meter(s)
m^3/s	cubic meter(s) per second.
mm.	millimetres.
M.W.L.	Mean water level.
N.	Newton.
NEDECO.	Netherlands Engineering Consultants.
N-value	standard penetration test value.
P.W.D.	Public Works Datum, all levels used are relative to P.W.D. which is 0.4599 meter below New Mean Sea level.
R.H.D.	Roads and Highways Department
R.P.T.	Rendell Palmer & Tritton Ltd.
R.R.I.	River Research Institute, Faridpur Bangladesh.
s	second(s).
S.H.W.(L)	Standard High Water (Level)
S.L.W.(L)	Standard Low Water (Level)
S.P.T.	Standard Penetration Test.
S.T.S.	Short Term Study; this refers to the Meghna River Bank Protection Short Term Study undertaken by Haskoning/Delft Hydraulics/BETS.

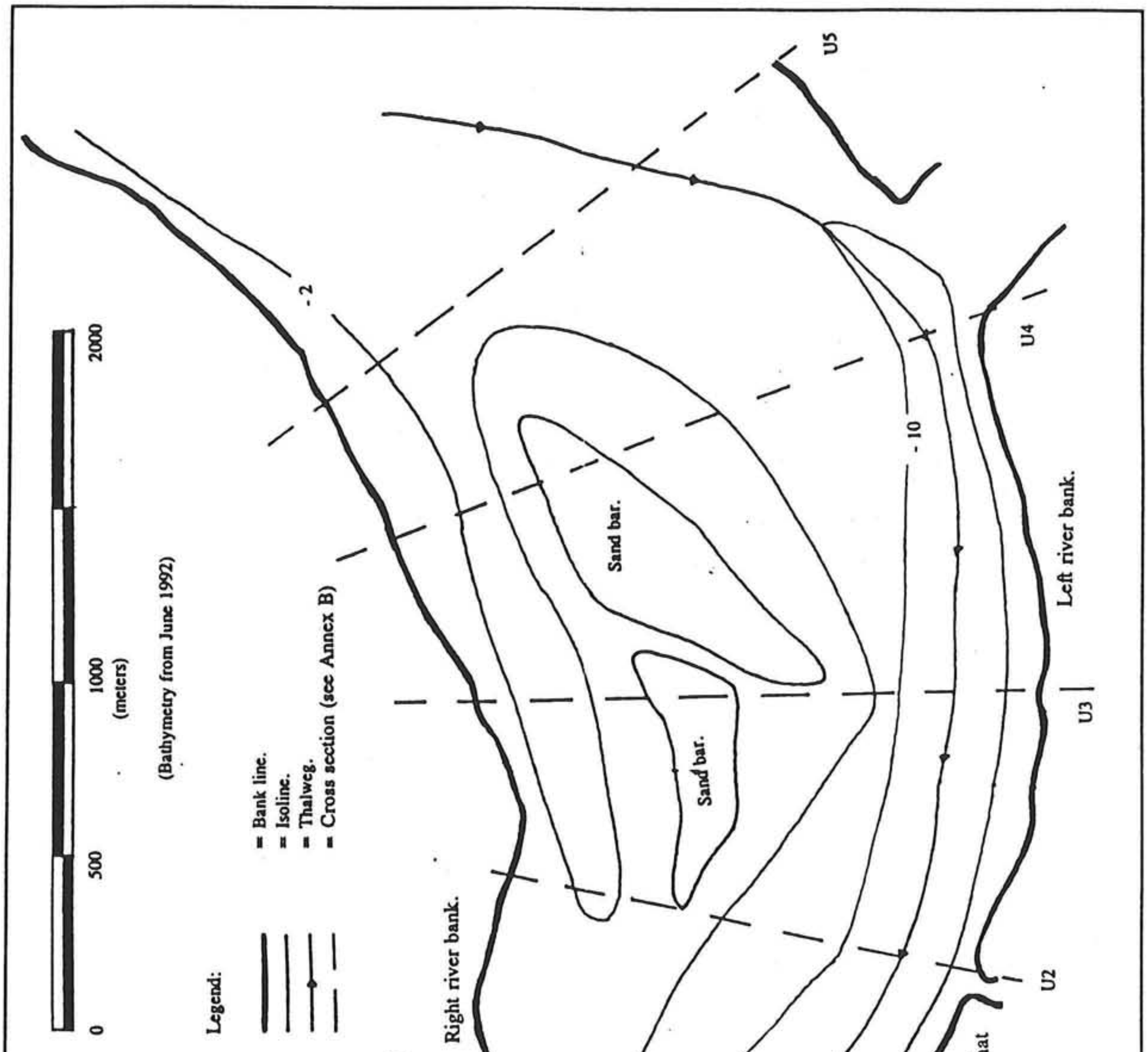


Figure i.1: Layout of the situation near the Meghna Bridge.

PART I: ANALYSIS AND DESCRIPTION OF THE PRESENT SITUATION

- Introduction:

In order to keep this Report in a readable format it is divided into two parts. In part I the field reconnaissance, data collection and the analysis of the present situation are discussed. Part II discusses the design study.

The more background information is presented in various Annexes.

The analysis and description of the present situation (part I of this Report) is composed as follows:

- Firstly, in Chapter 1, the origin of the project, its location, the problem description & setting and the scope & limitations of this study are discussed.
- In Chapter 2 the river system in Bangladesh, and especially the Upper Meghna River, are discussed.
- Subsequently, in Chapter 3, the design parameters for the project location are discussed. For the derivation of these parameters it is often referred to the various Annexes.
- As scour is an important phenomenon within this study this is discussed separately in Chapter 4 (and Annex F).
- Finally, in Chapter 5, the original design for bank protection works near the Meghna Bridge is discussed and a conclusion is drawn for part II of this Report, where the design study is discussed.

1 INTRODUCTION

1.1 Origin of the project

In February 1992 the Meghna River Bank Protection Short Term Study (S.T.S, ref [2]), undertaken by Haskoning/Delft Hydraulics/BETS was finalized. This study discussed possible bank protection works at critical locations along the Meghna River in Bangladesh. Along the Meghna seven locations were identified where measures for protection against erosion were needed. From these seven locations two priority sites were determined and studied to a detailed design level. For the other five locations a feasibility study was done.

One of those five remaining locations is the location of the Meghna Bridge which is studied in this project.

1.2 Project location

Two of the most important cities of Bangladesh, namely Dhaka and Chittagong, are connected by National Road 1 which has a length of approximately 286 km. This highway connects the political and economic centre (Dhaka) with the industrial centre and the country's largest trading port (Chittagong) and crosses the countries highest populated area.

Originally the highway crossed the Upper Meghna River, at a place where it is divided into two channels, using ferries. Due to the economic developments in the last decades however, these ferries were more and more becoming a bottleneck for the road transport system.

In 1960 a first attempt was made to improve the highway. It was proposed to construct bridges replacing the ferries. The implementation of this idea took a long time however.

In 1984 this project was adopted as a so-called "Japanese grant aid project" and in the same year a start was made with the feasibility investigations for the construction of two bridges, at the same locations as the existing ferries, by a Japanese consultant. In september 1987 the construction of the first bridge, the "Meghna Bridge"[3], was started. After finalization of this Bridge in 1990 a start was made with the construction of the bridge crossing the second channel of the Upper Meghna River; the so called "Meghna-Gumti Bridge".

The Meghna Bridge is located 28 km south east of Dhaka and has a length of 930 m. The Upper Meghna (both channels together) here has a discharge with a hundred years return period of 20,900 m^3/s. It is a meandering river which erodes its non protected banks. Upstream from the Bridge the former ferry ghats are situated.

The project location is depicted in Figure I.1.

A general layout of the situation near the Meghna Bridge is given in the Figure i.1 on Page x.

[3] The official name for the Meghna Bridge is "Meghna Roads and Highways Bridge". This name distinguishes the Bridge from the Meghna Railway Bridge, located about 80 km. more upstream at Bhairab Bazar. However, within this study the Bridge is simply called "Meghna Bridge" or "Bridge".

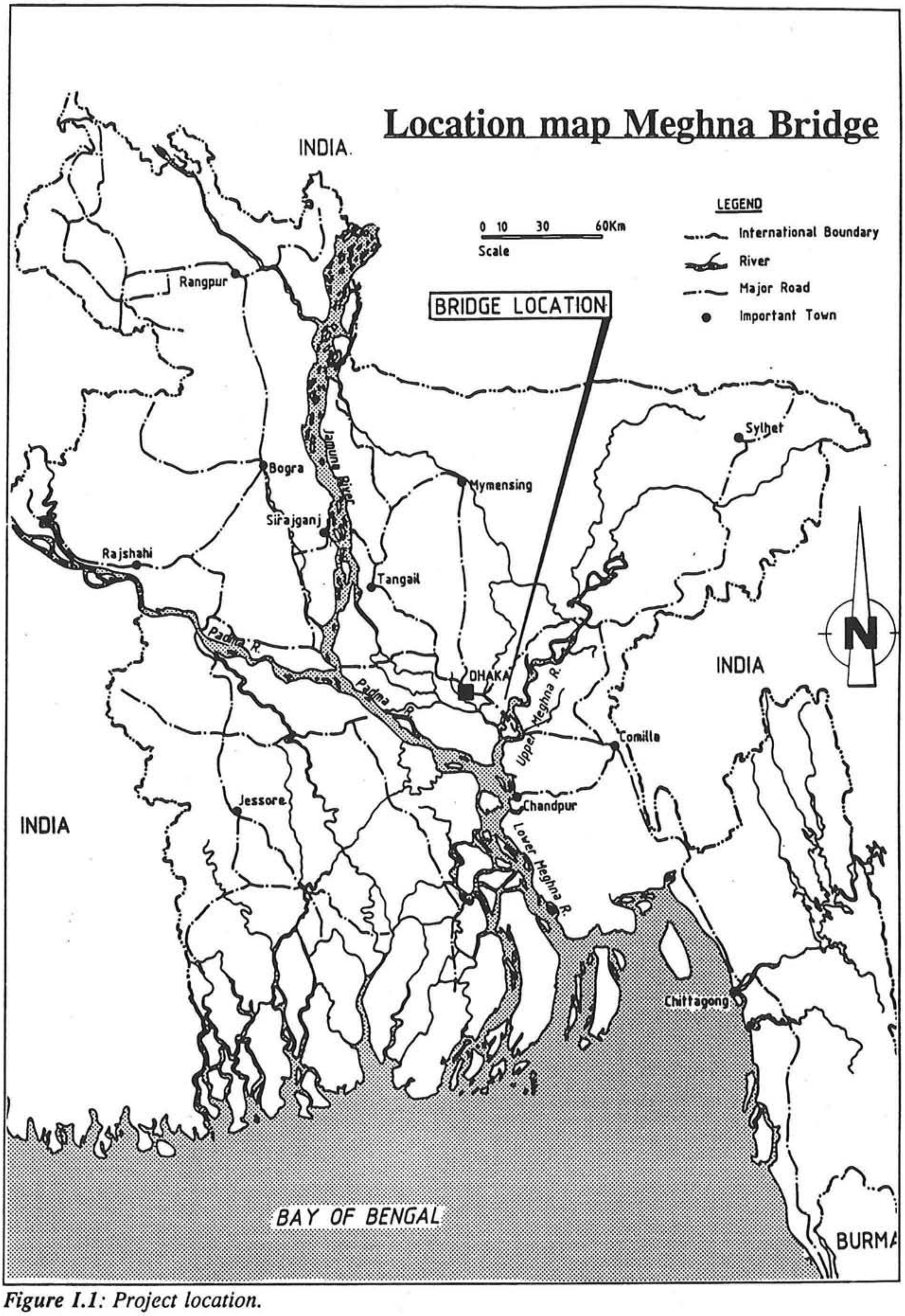


Figure I.1: Project location.

1.3 Problem description and setting

1.3.1 Problem description

For the design study for the Meghna Bridge no real elaborated river study was conducted. In the original design only small protection works for the Bridges' piers and abutments and a small "river training" work were foreseen. The items were the following:

- Protection of the piers against scouring by stone dumping.
- River bank revetments against erosion consisting of wire mesh gabions filled with stone placed on soil filled jute bags. The locations where the gabions and bags were to be placed extended on the Comilla side (left bank, outer bend) 450 meter upstream and 180 meter downstream along the river, while on the Dhaka side (right bank, inner bend) a 190 meter stretch downstream of the ferry ghat was planned.
- The small "river training" work; at the upstream side of the old ferry ghat at the Comilla side sheet piling with gabions in front was planned as a hard point safely guiding the flow under the Bridge.

During construction of the Bridge however, on december 30, 1989 a severe slope failure occurred along the river bank at the Comilla side, just upstream from the Bridge (for the location, see also Figure i.1), which damaged the left abutment severely. Afterwards the planned protection works were (a little) reviewed. The revised design of the left abutment consisted of cellular concrete/geotextile mattresses (see Annex A), with steel sheet piling at the toe of these revetments and placing of gabions in the river bed continuously in front of the sheet piles. This revised design is constructed along the newly formed river bank at the Comilla side.

At this moment however, two years after completion of the Bridge, caused by toe erosion, these concrete mats have broken and are sliding into the river (see also Annex A). This resulted in undertaken temporary protection works, consisting of more gabion dumping, to protect the Bridge's left abutment during the end of the 1992 flooding season. Also a study to take (temporary) safety measures to protect the Meghna Bridge is started by P.C.I. (see ref. [7]). This study by P.C.I., is focused only on protection of the Bridge itself so river course fixation is no part of it.

Two developments are important for this study:

1) The left bank abutment of the Bridge is located at an eroding bend (see Figure I.2). A very large sand bar is slowly advancing towards the left bank. The thalweg is situated near the left bank. As shown by bathymetric surveys, undertaken during the construction of the Bridge, the river bed changed substantially in 1988 (when there was a severe flood). The river bank erosion is, due to the shifting of the main channel (and thalweg) to the left bank, eroding the toe of the bank slope and causing small slope failures. The annual erosion reported in the last five years amounts to between 40 and 50 m per year.
   Downstream from the Bridge, at the right bank, also erosion is taking place. Comparison of satellite images from 1973 and 1990 revealed an erosion of 100 m in 17 years, downstream of the Bridge.
   In the Meghna River Bank Protection Short Term Study (Feb. 1992, ref [2]) a further erosion at the left bank upstream of about 15 m per year was predicted.

2) Concentration of the river flow, propagation of the scour hole in front of the former ferry ghat at the left bank and constriction of the cross sections by the construction of the Bridge together cause constriction scour of the river bed under and in the vicinity of the Bridge.

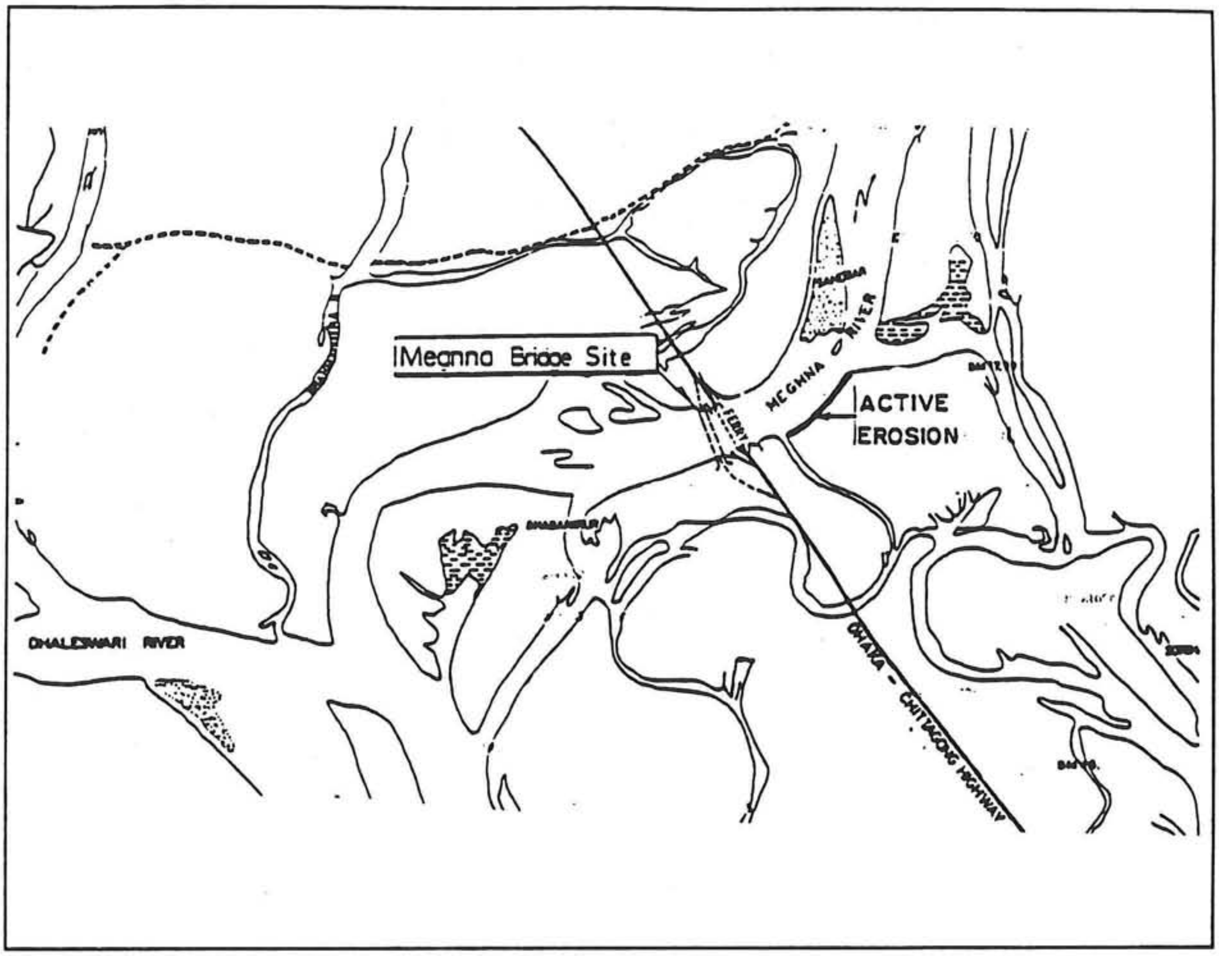


Figure I.2: Eroding bend upstream from the Meghna Bridge.

Also, due to protrusion scour, a deep scour hole is formed just downstream of the ferry ghat. This scour hole has reached the Bridge between the piers 8 and 9 (the numbering of the piers is shown later in Figure I.4). The revetments for the left abutment are already severely damaged by the undergoing slope failures.
During the 1992 flood temporary remedial works were undertaken to defend the Bridge's abutment. At the moment the piers of the Bridge are still assumed to be safe.

Flood control will not be part of this study.

1.3.2 Problem setting.

Regarding the story above, it is clear that the problems at the location of the Meghna Bridge can be divided into two parts:

1) The continuing erosion of the outer bend of the river upstream of the Bridge can lead to outflanking of the Bridge by the river.
2) The Bridge's left abutment (and piers) are attacked by deep scours. This has already severely damaged the abutments' revetments.

However, these two parts are not as separated as it may seem. The problem of outflanking, mentioned under 1) redirects the flow of the river in such a way that the left abutment is being attacked (and damaged). Therefore it must be borne in mind that solutions for these two sub-problems may be combined ones.

1.4 Scope and limitations

1.4.1 Scope

The scope of this study is to design bank protection works for the left bank upstream from the Meghna Bridge to protect the Bridge on the long term and to prevent the Bridge from outflanking by the River.
Former studies are analyzed and supplemented by information collected in Dhaka. Consequently a review on scour computations and predictions (because this is the core of the problem) is made and a detailed design is elaborated.

1.4.2 Limitations

The most important limitation is the fact that flood protection is not within the scope of this study. This means that top levels of the bank protection works are not taken higher than the level of the hinterland.

The design study (part II of this Report) only deals with bank protection measures. As it can be seen in Chapter 4 further protection of the Bridge piers is not necessary and therefore is not dealt with. The measurements taken at this moment to safeguard the piers are assumed to be sufficient.

2 THE UPPER MEGHNA RIVER

2.1 The river system in Bangladesh

Before discussing the Upper Meghna River itself (Section 2.2) here first a brief explanation about the river system in Bangladesh is given. This river system is schematically depicted in Figure I.3.

The river system in Bangladesh is dominated by three rivers: the Jamuna (Brahmaputra), the Padma (Ganges) and the Meghna.

- The Jamuna river

The Jamuna river originally rises at the north slopes of the Himalayas as the Tsang-Po River. After flowing through Assam (India), there being called the Brahmaputra, it crosses the Bangladeshi border from where it is called Jamuna. In Bangladesh, before the confluence with the Padma, the Jamuna is joined at its right banks by the Teesta and the Atrai while the Old Brahmaputra (which was the original course of the Jamuna some 200 years ago), the Arjam and the Dhaleswari river at the left bank divert themselves from the Jamuna.

The total river length is 2,880 km. and the catchment area upstream from Bahadurabad amounts to 573,500 km^2. Due to the melting of snow in the Himalayas the river starts rising in March, which causes a first discharge peak in May, early June. Subsequently peaks occur in July-August in response to heavy monsoon rains in Assam and Bangladesh. In 1988 the peak discharge was almost 100,000 m^3/s. The average annual discharge at Bahadurabad for the period 1964-1988 amounted to 19,462 m^3/s which is equivalent to a runoff of about 1150 mm. annually.

- The Padma river

The Padma river originates being called the Ganges on the southwestern slopes of the Himalayas (India). After crossing the Bangladeshi border, from where it is called Padma, and being joined by the Jamuna at Gualundo it flows on for 130 km before confluencing with the Upper Meghna River at Chandpur.

The total length of the river from its westernmost sources to Chandpur is 2,600 km. The total drainage area is slightly over 900,000 km^2. The river starts rising gradually in May-June and attains its peak on average early September. Peak flows up to 76,000 m^3/s (1987) have been reported. Although the peak flows on the Padma are experienced on average about one month later than on the Jamuna, there is a high probability that both phenomena coincide. The average flow at Hardinge Bridge of 10,874 m^3/s (1964-1988) implies a runoff of 380 mm, which is considerably less than that of the Meghna and Jamuna rivers.

- The Meghna river

The Meghna river can be divided into the Upper and the Lower Meghna. The Upper Meghna will be discussed hereafter (Section 2.2). The Lower Meghna river starts from the confluence of the Upper Meghna River and the Padma at Chandpur. The Lower Meghna conveys the melt and rain water from the Padma, Jamuna and Upper Meghna basins to the sea.

The total catchment area is about 1,637,000 km^2 and the maximum discharge can be as high as 155,000 m^3/s. The major contribution of the discharge originates from the Jamuna river (annual average 19,462 m^3/s) and the Padma river (annual average 10,874 m^3/s).

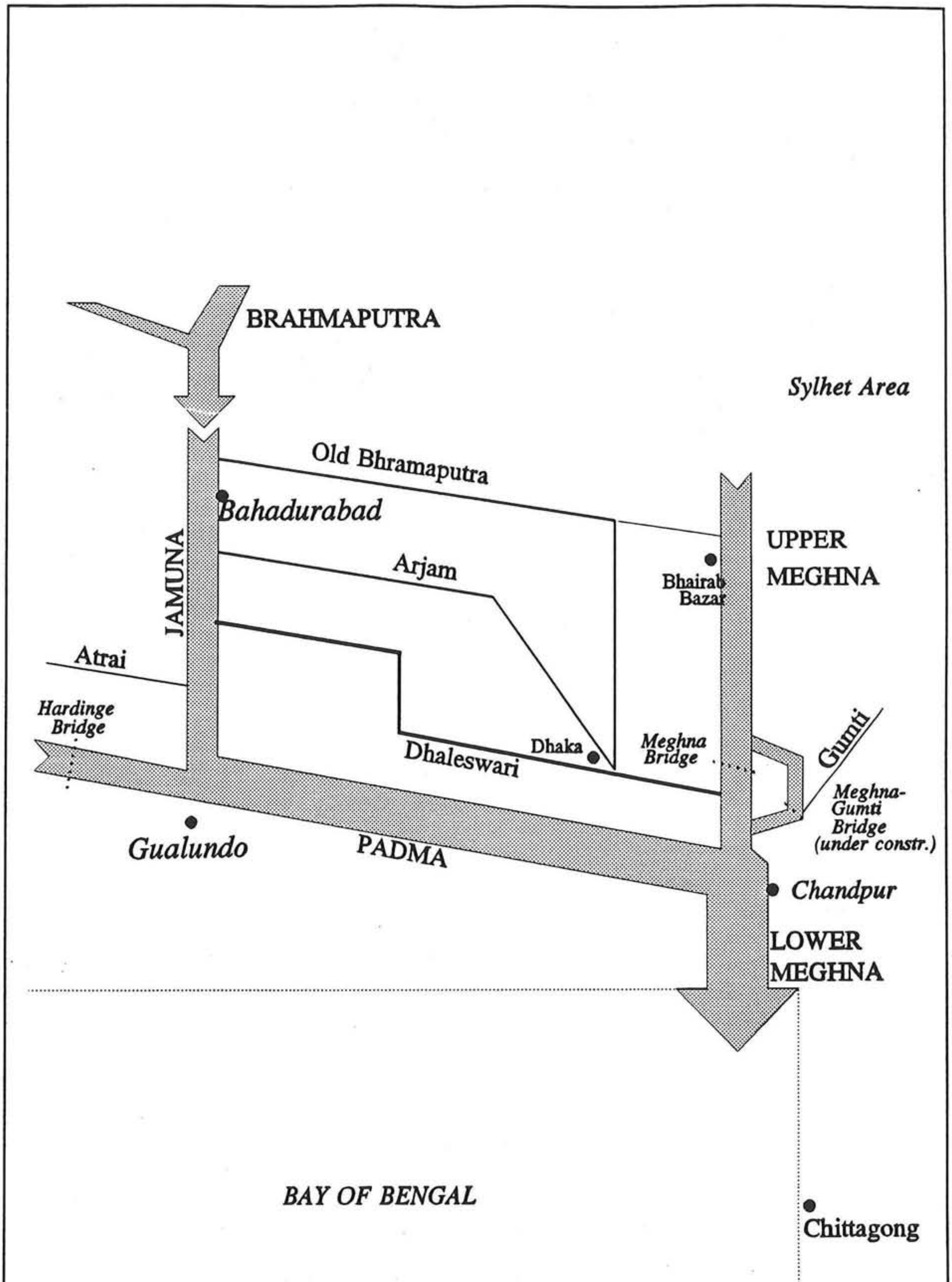


***Figure I.3**: Schematic layout of the river system of Bangladesh.*

2.2 The Upper Meghna River

The Upper Meghna River carries the combined flow of the Boulai, Surma and Kusiyara Rivers which originate in the Indian hills north east of Bangladesh. The Surma River flows through the Sylhet area which, on a geological scale, is rapidly sinking away. The sinking amounted 9-12 m over the last two centuries. Some tributaries of the Surma originate from the Sisang hills in India and from piedmont areas. They bring quite some sediment (boulders, gravel, sand) that most probably settles in the Sylhet area. Between Bhairab Bazar and the Meghna Bridge no significant contributions to the discharge, such as confluences, occur.

The Upper Meghna River drains an area of 77,000 km^2 of which about 46,500 km^2 is located outside Bangladesh. At the Meghna Bridge the drainage area contains 69,890 km^2. Although having some reaches with a system of various channels, the Upper Meghna can be characterized as an alluvial river meandering within a well defined high water bed and having flood discharges up to around 20,000 m^3/s. At the chainage of the Meghna Bridge the Upper Meghna River has two channels from which the more important one with around 2/3 of the total discharge flows under the Meghna Bridge. To be able to cross also the other channel more easily a second bridge (the so called Meghna-Gumti Bridge) is now under construction.

2.2.1 River geomorphology.

Here the river geomorphology of the Upper Meghna River in general is discussed.

- Planform characteristics

Though seeming to be a braided river the Upper Meghna is better classified as a meandering river. Studying BWDB cross sections from the Upper Meghna it becomes clear that the Upper Meghna consists mainly of one single channel and over two reaches (one of them being the reach in which the Meghna Bridge is located) over two parallel ones of almost similar importance. All other channels which can be seen clearly on satellite images carry too little water to be of any importance. These images also reveal many abandoned rivers or rivers which carry no water during the low flow season. The explanation for this is probably that the Upper Meghna River flows in a bed that was shaped originally by the Brahmaputra River and that the present river is still in the process of adapting its bed to the new conditions.

- Historical planform changes

To understand the behaviour of the Upper Meghna River and its characteristics and to be able to predict future developments some knowledge about the historical planform changes is required.

The most important historical development is the change of the Brahmaputra (Jamuna) from its former to its present course. About some 200 years ago the Brahmaputra has changed its course more to the west thereby abandoning the Upper Meghna River. Though not much is known about the exact events during this process one result is very clear; at present the Upper Meghna River is flowing through a river bed which is formed by a river with a maximum discharge of around 100,000 m^3/s instead of the 20,000 m^3/s maximum discharge of the present river. From several studies it is concluded that the Upper Meghna River still is adapting itself slowly to this new situation.

The more recent changes in the planform, which are very important for the Bridge location, are discussed in Annex D.

2.2.2 General characteristics

The general characteristics of the Upper Meghna River, as a result of measurings conducted during the S.T.S., are presented here briefly. It is emphasized that these parameters are valid for the Upper Meghna being one single channel and not automatically for the location of the Meghna Bridge, where the main channel is split. A detailed analysis of all the parameters involved in this study, specified to the location of the Meghna Bridge, is given in Chapter 3.

Due to the historical planform changes the Upper Meghna River has some deviating characteristics. Most of the river characteristics are more or less constant upstream from the confluence with the Dhaleswari. The characteristics discussed below are valid within this stretch.

- Hydrology

The average daily water levels at the Upper Meghna River vary from 0.5 to 7.0 m+PWD.

Since the river bed gradient is extremely gentle at about 1 to 50,000 tidal influences affect even river stretches upstream from the Meghna Bridge location when the water level is low (dry season). The maximum tidal range at the Meghna Bridge Site varies from about 0.25 m (Aug-Sept) to 0.80 m (Feb-Mar). The combined maximum discharge through both channels together at the Meghna Bridge is about 20,900 m^3 /s.

- Channel dimensions

The average cross-sectional area at bankfull discharge is about 10,000 m^2, while the average bankfull width is some 1,200 m and the bankfull depth varies between 7 and 14 m, with an average of about 10 m.

- Hydraulic parameters

It is remarkable that even for flood conditions the water level slopes in the Upper Meghna River are very small, in the order of 1.8 m per 100 km ($\approx 2.0 * 10^{-5}$) which is much smaller than the present slope of the Jamuna River ($\approx 7.0 * 10^{-5}$) which occupied the valley of the Upper Meghna River some 2 centuries ago.

At the Upper Meghna River no flood embankments are present. The bankfull discharge corresponds to about 8,000 m^3/s, a value which is exceeded about 27 % per year. This is very often. During this time the hinterland is inundated.

- Bed material

The average values of the characteristic particle sizes are:

D_{16} = 0.037 mm,
D_{50} = 0.14 mm and
D_{84} = 0.25 mm.

- Sediment transport

The dominant mode of transport is suspended load.

With the Engelund and Hansen formula the predicted sediment transport rate for a discharge of 10,000 m^3/s is 0.68 m^3/s which is equivalent to 6 ppm (very low). The range of measured sediment concentrations is also very low.

The estimated yearly sediment transport is around $1.3 * 10^6$ ton. This is only about two promille of the yearly sediment transport in the Jamuna River.

The dominant discharge, defined as the discharge carrying most of the sediment, is about 11,000 m^3/s.

The Rhine, which is also a river carrying few sediment, has a yearly sediment transport of about $0.72 * 10^6$ ton but with an average discharge of 2.200 m^3/s instead of the approximately 6.000 m^3/s average for the Upper Meghna River.

2.2.3 Conclusion

The image emerges of a river with a very gentle slope and therefore carrying hardly any sediment. Already in the upper catchment area (the sinking Hoar area) all the sediments settle down. Hence at the location of the Meghna Bridge no sediment is left in the Upper Meghna River.

Within the S.T.S. it was revealed that channels of the Upper Meghna River which are relatively unimportant, regarding their average discharge, do not seem to be silting up. This must also be due to the fact that the Upper Meghna River carries very little sediment so there is also few sediment available for deposition.

The fact that the bankfull discharge is exceeded so often may be due also to the low sediment transport rate which is probably not able to raise the floodplain level with the same celerity as the ground levels are subsiding.

In Annex C, where the hydraulic and hydrological data analysis at the Meghna Bridge Site is discussed, it is concluded that it is impossible to derive stage-discharge or stage-flow velocity relations here. This must be due to the fact that the water levels and discharges in the Upper Meghna River are mainly governed by its lower boundary which is the confluence with the Padma River.

3 DESIGN PARAMETERS FOR THE MEGHNA BRIDGE SITE

3.1 Introduction

In this Chapter the local conditions at the Meghna Bridge Site are formulated. This is done by analyzing all the available data from former studies and completing them with the data collected in Dhaka. For some topics however, the design parameters, as derived in the Short Term Study, are unchanged because no additional data, which could have changed the insights, became available in the meantime. This is especially the case for the geo-technical characteristics.

For the derivation of the design parameters no account is taken for possible future nation wide flood protection measures. It is also assumed that no other major changes in the river system will occur.

A description and evaluation of the presently existing bank protection works is given in Chapter 5.

After discussing the return period to be applied for design purposes in the following Sections the following subjects will be discussed:

- 3.3: Hydrology/hydraulics.
- 3.4: Geo-morphology.
- 3.5: Geo-technics.
- 3.6: Available construction materials.
- 3.7: Costs.

Within these Sections is referred to the various Annexes which discuss the subjects more extensively.

With regard to the levels used it is remarked that they are all related to Publics Works Datum (P.W.D.) scale.

3.2 Applied return period for design purposes

Within the Short Term Study it was proposed to conform to the local economic approach for the design of bank protection works. This approach prescribes to design protection works with a safety for a 50 years return period for rural areas and for a 100 years return period for urban areas and important infrastructure.
As the Meghna Bridge is part of one of the most important highways of Bangladesh a return period of 100 years will be used for design purposes. However, this does not mean that the top of the bank protection construction is taken at a water level with a 100 years return period. As the works are not meant for flood control purposes the top of the bank protection is not chosen higher than the hinterland (around 6.00 m+PWD). The strength of the works however, is designed based on a return period of hundred years.

3.3 Hydrology/hydraulics

The data analysis is presented in Annex C. Here only the final results are repeated. The values mentioned are valid for the location of the Meghna Bridge, i.e. only the right branch of the Meghna here, and are used for design purposes.

- The water level with a return period of 100 years is 6.59 m+P.W.D.
- The discharge for a return period of 100 years through the channel of the Meghna Bridge is 13,600 m^3/s.
- The maximum flow velocity with which micro slope surface stability analyses are elaborated is 1.6 m/s.
- The level duration curve is presented in Annex C (Figure C.4).
- The discharge duration curve for the location of the Meghna Bridge can be derived from the curve for Bhairab Bazar, as derived for the S.T.S., using a multiplier of 0.72.
- The maximum piezometric head difference over the embankments is 3.5 m.
- The design wave height is the wave height for wind waves which is 0.98 m for a return period of 100 years.
- The bankfull parameters for the Meghna Bridge Site are:
  - Bankfull area : 11,000 m^2
  - Bankfull width: 900 m,
  - Bankfull depth: 12.5 m.
- The dominant discharge is 8,000 m^3/s, with an assumed water level of 4.1 m+P.W.D.

3.4 Geo-morphology

In Annex D the geo-morphological characteristics for the location of the Meghna Bridge are discussed. This resulted in adopting Figure D.4. as a stable planform and as a basis for the design of protection works thereby taking in account the danger for outflanking of the Bridge by the river. For further details concerning the geo-morphological history of the Upper Meghna River, see Annex D. In addition to this Annex the local scour locations are discussed below.

- Local scour locations

During the site visits and from measurements conducted by the Japanese Bridge contractor the local scour locations were identified. Presently the deepest scour holes are located around the ferry ghat (at the left bank of course) and between several Bridge piers (especially piers no 6, 7, 8 and 9) which are in the main flow area. This is shown in Figure I.4.
The lines are a result from a river bed survey just outside the Bridge axis in July 1990 and June 1992. Though attacked, the scour around the Bridge piers doesn't (yet) seem to be a danger for the Bridge stability. Small emergency works were undertaken (more stone dumping) and for the moment the safety of the piers appears to be sufficient. However, intensive monitoring of scouring processes around the Bridge's piers in the future is very important.

Cross sections from measurements at the deepest scour hole in front of the former ferry ghat are shown in Figure I.5.

Scour calculations and the situation at the location of the Meghna Bridge are discussed in Chapter 4, 5 and Annex F.

3.5 Geotechnics

In Annex E the geotechnical data analysis is presented. In this analysis the geotechnical design parameters and the safety coefficients for macro slope stability analyses are determined. Here these results are not repeated. In addition to this Annex however the slope failure, as occurred in 1989, is summarized below.

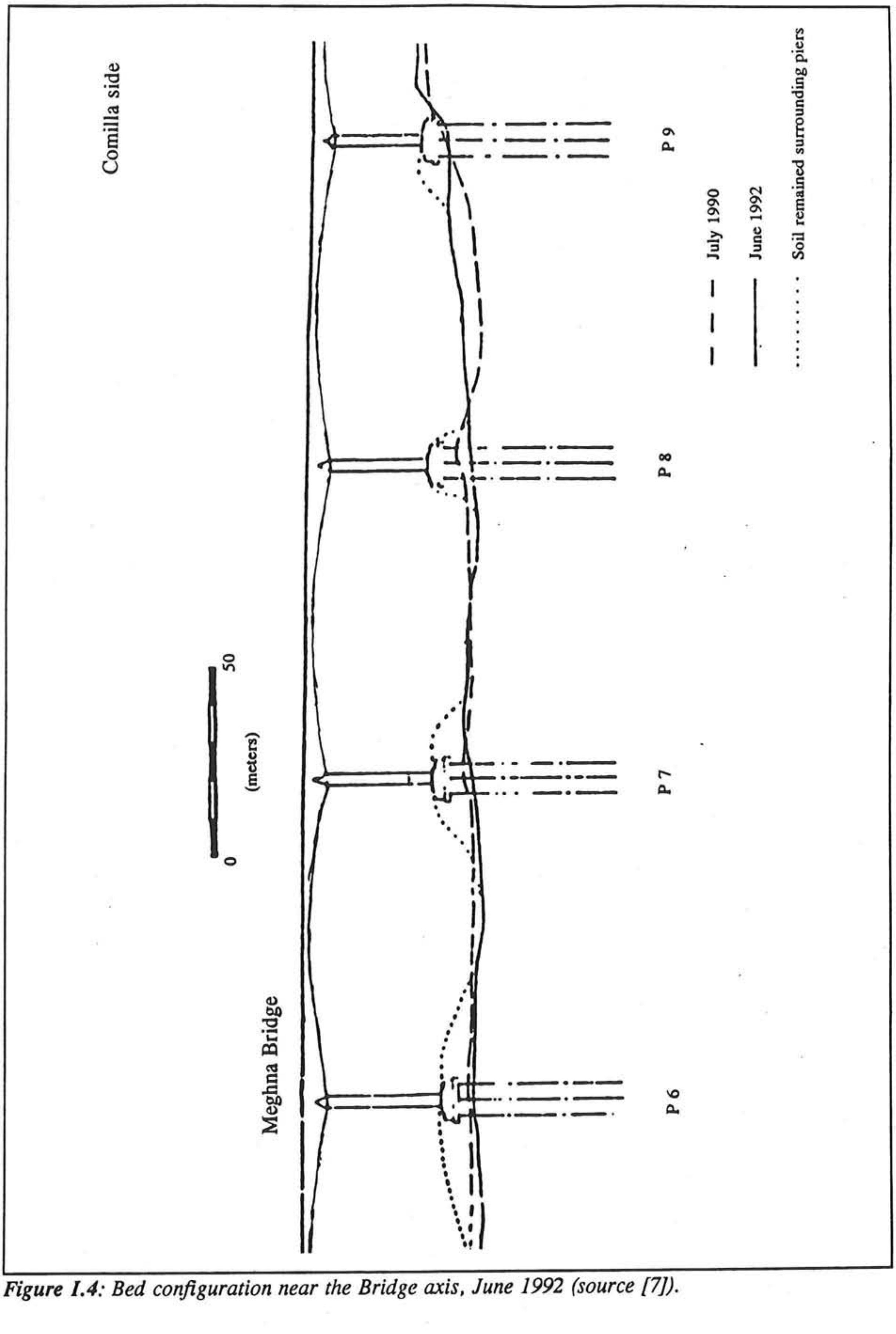


Figure I.4: Bed configuration near the Bridge axis, June 1992 (source [7]).

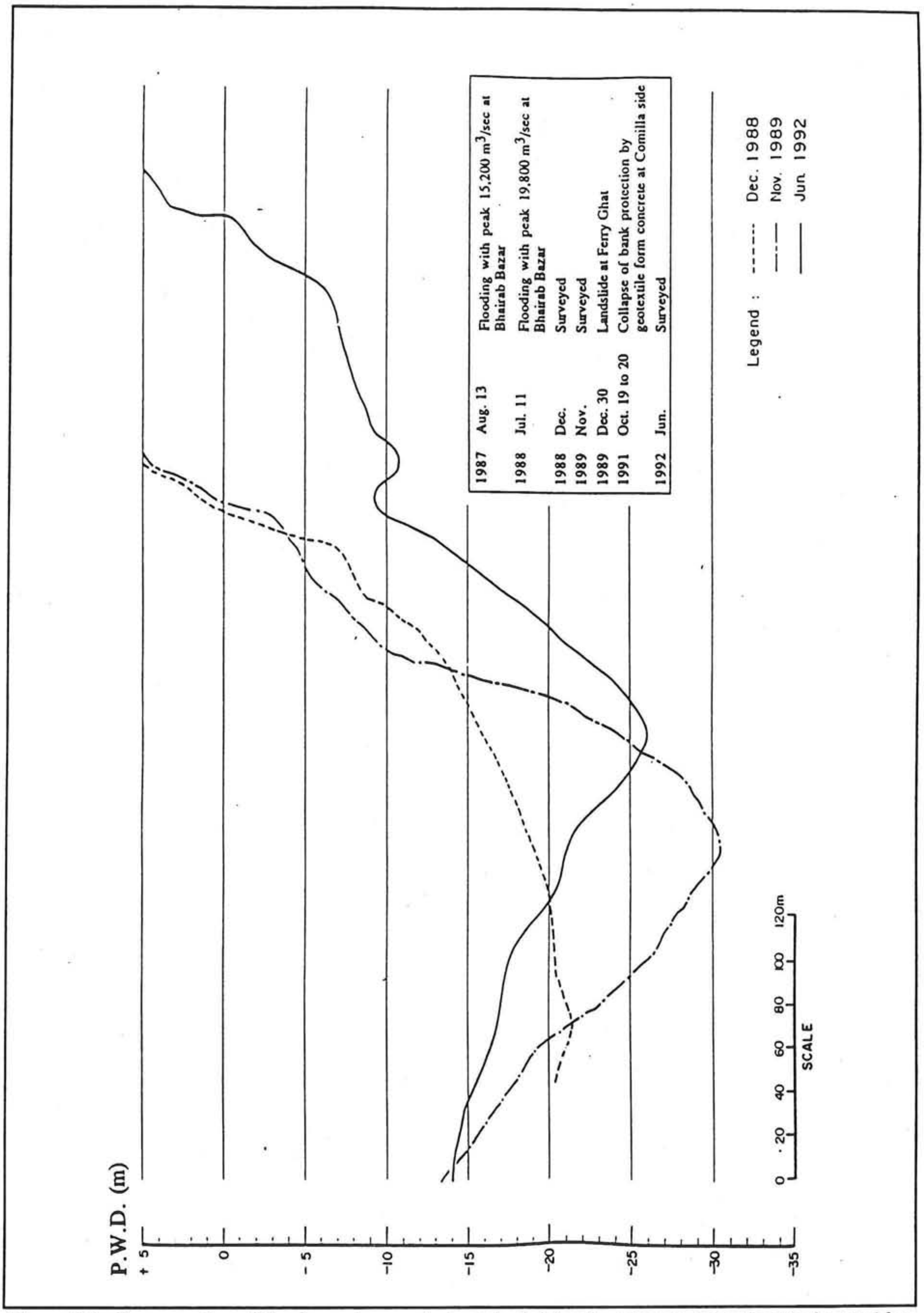


Figure I.5: Cross section of the deepest scour hole at the left bank upstream from the Meghna Bridge.

On December 30th, 1989 a severe slope failure occurred along the left bank just upstream from the Meghna Bridge (under construction). The mechanism causing this phenomenon was probably the following:

The shifting of the thalweg to the left bank caused a heavy erosion and therefore steep (maximum) bank slopes. From Figure I.5 the following bank slopes can be derived:

***Table I.1**: Bank slopes occurring at the left bank upstream from the Meghna Bridge.*

Date	Average (V:H)	Maximum (V:H)
Dec. '88	1 : 9.5	1 : 2.4
Nov. '89	1 : 4.6	1 : 2.1
June '92	1 : 7.0	1 : 3.2

At the end of 1989 the average slope of the unprotected river bank was around 1 : 4.6 (V:H). However, locally bank slopes of 1 : 2.1 (V:H) occurred. This in combination with a reasonable ground water gradient (not measured) after the big flood must have caused the slope failure.

Another mechanism is not very probable as the recorded water level and flow velocity were only 1.20 m+PWD and 0.40 m/s at Meghna Bridge. The drawdown speed of the water level is estimated to be around 5 cm/day during the second half of december 1989. From Table I.1 it can be seen that after the slope failure both the average and maximum slope steepness has diminished

3.6 Available construction materials

From the Short Term Study a brief analysis of the locally available construction materials is available. It needs no explanation that using these materials is highly preferable.

- Sand

Delivery of sand (e.g. for the production of concrete) will not be a problem. Often a mixture of 'local' and Sylhet sand (which is coarser) is used to get a proper grading. All sand in Bangladesh has a rather high mica content, and often a high percentage of fines ($d < 0.063$ mm.) is present.

- Boulders

Within Bangladesh 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 about 100.000 m^3 of boulders can be delivered to the site within six months, but not without certain problems. Large orders disturb the market equilibrium and force prices upward. Boulders can be used for protection of both the upper and lower parts of the slope (e.g. single boulders, gabion mattresses or sack gabions).

- Cement

Cement can be required for the production of concrete or cement blocks and is available from local sources in Bangladesh. The production is limited, however.

- Rock

Within Bangladesh the only place where suitable rock is found at or near surface level is in the Chittagong Hill Tracts. The existing infrastructure and security situation can probably make it difficult to obtain large quantities of rock in a short period from this area. The only other way is to import it from, for instance, India or Malaysia. From India the rock would have to be transported by train (± 1000 km.) and from Malaysia (± 2500 km.) the required grading could easily be obtained from existing quarries and transported by barge to Bangladesh.

- Bitumen

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

3.7 Costs

It is impossible and also not practical her to list a series of goods and their prices. But some general remarks can be made anyway.

With Bangladesh being a developing country mechanised labour is relatively expensive while manual labour costs are very low. Also importing construction materials is very expensive compared to the cost of local materials. Therefore designs consisting of local materials and labour are highly preferable.

Preliminary cost estimates are elaborated for the final designs in the design study.

4 SCOUR

4.1 Introduction

In this Chapter at first general scour for the location of the Meghna Bridge is discussed. Subsequently the general characteristics of the local scour types are briefly introduced and discussed.

In Annex F a general description of models and formulae to derive local scour is given. Also an application of these to the location of the Meghna Bridge is elaborated and the design scour for the various alternatives (see part II of this Report) is derived. Therefore in this Chapter, when local scour is concerned, only briefly the types, which are applicable to the location of the Meghna Bridge, are discussed. For all scour calculations is referred to Annex F.

4.2 General scour

(For this subject mainly the S.T.S. is quoted)

General scour is the response of a river to changes in the river itself and in its boundary conditions and must be taken into account when designing bank protection works in an affected reach.
Recently the Upper Meghna River has not been subjected to major changes in the river itself. There have been changes (see also Annex D) over the past decades and centuries however:

i. The shifting of the Brahmaputra River to its present (Jamuna) course, leaving the Upper Meghna River as an oversized channel system.
ii. The sinking away of the Hoar (= Sylhet) area which is probably continuing also now.
iii. Accelerated soil erosion in the catchment area.

Future developments which can be foreseen at present are:

iv. Flood embankments.
v. Sea level rise.
vi. An increased diversion of water for irrigation from the Upper Meghna River.

The influence of these factors is discussed briefly hereafter, especially as far as the effect on bed levels is concerned:

ad i. Shifting of the Brahmaputra River.
The "scars" of the Brahmaputra river had to be filled up, and that will have caused aggradation. Whether this process is still continuing cannot be ascertained but the deviation of the present planform of the Upper Meghna River from a meandering course suggests that this aggradation might still go on.

ad ii. Sinking away of the Hoar area.
This may have a substantial influence on the Upper Meghna River. However, data are missing to evaluate this effect.

ad iii. Accelerated soil erosion in the catchment.
Most of the possibly increased sediment yield from the Tripura will deposit in the Hoar region and hence will have a negligible effect on the downstream reaches. There may be some effect on the Kushiara River.

ad iv. Flood embankments.
These will cause an increase of the bankfull discharge and increased depth of the main

channels. The effect will be scour. As there is no real plan to construct flood embankments yet this effect will not be taken into account.

ad v. Sea level rise.

The possible sea level rise is estimated being about 0.5 m in the year 2050. The effect of this sea level rise will be a tendency for aggradation in both the Lower and Upper Meghna Rivers.

ad vi. Increased diversion of water for irrigation.

This may cause a slight increase in the slope and aggradation in the river. This diversion is only substantial in the low flow season.

For the Jamuna Bridge studies the effect of "outward" controls have been studied using an one-dimensional mathematical model of the river morphology. It was found that the sea level rise is dominant over the other possible causes also because of the short distance between the sea and the Jamuna Bridge. With the Meghna Bridge being even nearer to the sea a negligible effect of the other possible causes can be estimated. As the sea level rise will occur only in the coming decades, general scour will not be taken into account for the design of bank protection works. Neglecting this effect results in a safe design approach as it is aggradation which is expected here.

4.3 Local scour

The title "local scour" is used here to emphasize the difference between these scour types and types of general scour.

In a future situation confluence scour, protrusion scour and bedform scour are not of interest. Therefore only the following types of local scour are discussed here:

- constriction scour,
- bend scour and
- local (or structure) scour.

4.3.1 Constriction scour

Constriction scour occurs if the river is constricted over a substantial length. For the present situation this mainly applies to the sitation in the vicinity of the Meghna Bridge.

The calculations, as elaborated in Annex F, reveal that the application of constriction scour models are only valid at the location of the Meghna Bridge itself, where the constriction is rigid. Applying the model for constriction scour to other cross sections results in major deviations compared to the levels actually measured.

4.3.2 Bend scour

Bend scour is the scour that develops in an outer bend and is due to the helical flow which occurs in bends. As the channel of the Meghna Bridge is a sequence of meanders bend scour occurs almost at the entire reach.

In the calculations in Annex F the bend scour is based on the design graph derived for the S.T.S. (Figure F.4.) The ratio's for $h_{bend}/h_{average}$ here vary from 1.15 to 1.50. Comparing these values to the data from the bathymetric surveys available they appear to be reasonable.

4.3.3 Local (structure) scour

Structure scour is the scour induced by man made structures in the river and is mainly due to the difference in roughness of the structures' revetment and the (alluvial) river bed. Sometimes also for specific structures (e.g. groynes) account is taken for the protrusive character.

While in other studies this type of scour is also called "local scour" this is not done within this study. To avoid possible confusion this type of scour here is called "structure scour".
The calculation models for structure scour are very simple and empirical. For the design studies for river bank protection in Bhairab Bazar scale model investigations were performed. The results for local scour(along revetments) were consistent with the calculation model. For this study it is assumed that this calculation model is also valid for the location of the Meghna Bridge.

4.4 Applied models in former studies

- In the Japanese studies for the design of the Meghna Bridge only scour computations were elaborated to estimate the scour around the Bridge piers. Using Lauren's and Lacey's formula this resulted in predicted scoured depths of 8.42 and 9.70 m (bottom levels of 19.42 m-PWD and 20.70 m-PWD). For these formulae an estimated bottom level of 11.00 m-PWD during flood periods was assumed. As in this project the Bridge piers are not concerned these models are of no further interest.

- In the S.T.S. for the location of the Meghna Bridge no scour computations were elaborated for the preliminary design. A scour depth of 10 m relative to the present river bed was adopted for the (preliminary) design of bank protection works in between the former ferry ghat and the Bridge.

4.5 Measured scour depths

For the Figures with the measured scour depths is referred to the Figures from Section 3.4. (Figure I.4 and I.5)

The deepest scouring points measured in November 1989 and June 1992 are 30 m-PWD and 26 m-PWD respectively. These are not only a result of the scour types mentioned above but also from scour induced by the former ferry ghat (protrusion scour). It can be assumed however, that in a future planform, when bank protection works are constructed the bank alignment will be smoothened and therefore, with the protrusion scour type not being present any more, scour depths will decrease.

At the Bridge piers the final scour depths were somewhat deeper than the ones accounted for (the deepest scour level in June 1992 was 22.00 m-PWD). This is dealt with by dumping more rock around these piers. However, this phenomenon is not expected to be endangering the Bridge.

4.6 Conclusion

It is known that the estimate of scour is a difficult matter. Scour estimation formulae are highly empirical and mostly derived for specific circumstances. For a good design however, scour estimates are very important.

In Annex F it is further explained how within this project the design scour levels are derived with the data available.

5 EVALUATION OF THE JAPANESE DESIGN & THE PRESENT CONDITIONS NEAR THE MEGHNA BRIDGE

5.1 Introduction

In this Chapter the original Japanese design for the bank, abutment and pier protection works at the left bank and its adjustments through time are discussed.

5.2 Original design

Within the design study of the Meghna Bridge not a real elaborated river study was foreseen. River engineering aspects were more treated following an "trial and error" approach. For the future a more detailed study concerning these aspects was foreseen. Here the design as proposed in the original design reports is discussed.

At the left bank the planned protective river works for the Bridge consisted of the following.

i. Protection of the piers against scouring.
ii. Revetments along the (left) river bank.
iii. Protection works around the former ferry ghat (left bank).

Within the design these items were part of the so-called "ancillary" works. Another important item which was part of the main study is:

iv. Design of the Bridge abutment and the approach road embankment.

Below a description of the original design of these four items is given.

- Protection of the piers against scour

For the design of protection works for the Bridge piers against scour estimates for scour were elaborated. Using Laursen's and Lacey's formulae the estimated scour depths (levels) were 8.42 m (bottom level: 19.42 m-PWD) and 9.70 m (bottom level: 20.70 m-PWD) respectively. Using these results it was decided to dump stones with a minimum size of 10 cm in an area around each pier of 600 m^2 with a layer thickness of 1.0 m.

- Revetments along the (left) river bank

It was planned to construct river bank protection 450 m upstream (which is until the upstream head of the former ferry ghat) and 180 m downstream along the left bank. This bank protection consisted of soil filled jute bags covered with stone. The minimum stone size required was calculated to be 65 cm (it was not noted whether this stone size is a D_n or D_{50} or other) to resist the expected wave heights of 1.75 m (also the estimate for the wave heights was not further explained). Therefore it was planned to use cylindrical wire mesh gabions filled with stones. These gabions would be fixed using wooden piles having a length of 1.5 m.

- Protection works around the former ferry ghat (at the left bank)

At the upstream head of the former ferry ghat over a total length of 78 m it was planned to drive steel sheet piles with a length of 30 m in the bank. In front of these sheet piles gabion placing in an estimated area of 1600 m^2 was foreseen.

- Design of the Bridge abutment and the approach road embankment

At the left bank side it was decided to construct two extra approach span bridges with a total length of 50 m (in fact this is already the first change in the original design). This was done considering the future development of bank erosion. Erosion of the left bank was measured with an interval of one year and it was concluded that the river moved five meters. Consequently it was concluded that 50 meters had to be sufficient to deal with the bank erosion during several years during which a detailed study of the river behaviour could be completed and bank protection measures, if necessary, could be taken up.
The road embankments were designed with a side-slope of 1:2 (V:H). Where these embankments exceed a height of 7.00 m+PWD a berm was foreseen at this level to ensure stability.

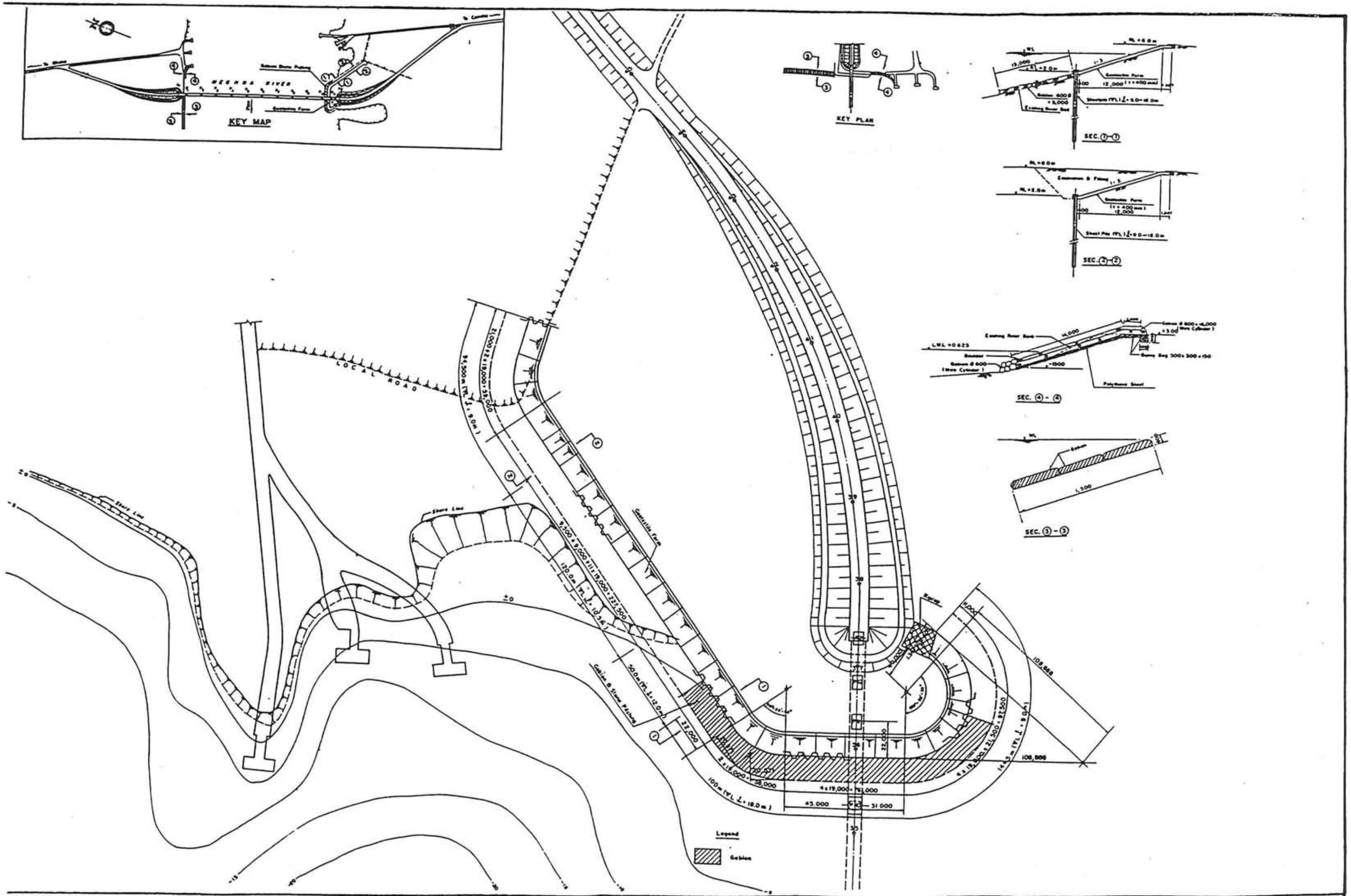
At the Bridge abutment this design was more extended. To protect the Bridge abutments from erosion it was proposed here to construct the slope of 1:2 (V:H) covered until the top with concrete blocks on crushed brick.

5.3 As-built situation

As mentioned before during the construction of the Meghna Bridge a severe slope failure at the Comilla side (left bank), between the Bridge abutment and the former ferry ghat, occurred at the end of 1989. This slope failure created a new river bank alignment. Also the floods in the years 1987 and 1988 were quite extreme ones. Realizing that the river power might have been underestimated the original design was further revised. At that moment not much of the river works was constructed yet. These design changes created substantial differences between the originally planned and the as-built situation. The following changes were made:

- The alignment of the river bank revetments was changed along the newly formed river bank and the revised design was to use a geotextile-form method consisting of concrete mats placed in geotextile form with steel sheet piling at the toe of these mats and placing of gabions continuously in front of the sheet piles. At the downstream side of the abutment, assuming less wave attack, rip-rap was foreseen as protection material. The details are shown in the Figure I.6.
- After changing the alignment nothing was mentioned about the planned protection at the head of the former ferry ghat. The site visit in september 1992 revealed that nothing was constructed here.
- The planned concrete brick surface layer at the abutment slope was changed into cellular concrete/geotextile mattresses.
- All slopes at the abutment, along the embankment and along the river banks which originally were foreseen to be 1:2 (V:H) were changed into 1:3 (V:H).

The protection of the piers against scour, as proposed in the design study, remained unchanged.



rawing left abutment of the Meghna Bridge.

5.4 Present situation

Here with "present situation" the situation as seen during the site visits, in September 1992, is meant.

- Effectiveness of the protection works

As mentioned already in the first Chapter of this Report the protection works, as constructed within the Bridge construction programme, appeared not to be sufficient. Eight months after the completion of the slope protection works, in October 1991, parts of the protection works collapsed. In Figure I.7 the situation of June 1992 compared with the original structures is shown. From Figure B.1. (Annex B) where the orientation of the abutment can be seen in a larger layout it becomes clear that especially the cross sections 2 to 6 are attacked by the river flow. This flow causes substantial erosion of the river bed in front of the structure, under the gabions. These gabions do not retain the underlying soil however. Because of the erosion the gabions shift downwards. A translation of these gabions to the centre of the river bed is not (yet) remarkable.
In some of the cross sections of the Figure (i.e. 2,4 and 5), because of the decreased support of the eroded soil in front, the non-anchored sheet piles collapsed thereby admitting small slope failures. The collapse of the structure at cross section 9 is probably due to the eroding forces of the turbulent flow patterns caused by the pier (no 10).

The pier protection as designed and constructed seems to be effective (see Figure I.4). The protection was designed for a lowest bottom level of 20.70 m-PWD. As mentioned already more material was dumped at the piers where this level is lower, but no real dangers are expected. Therefore, as mentioned in Chapter 3, within this study no more attention will be given to this aspect.

- General

From the site visits it became clear that the erosion of the left bank in the bend upstream of the constructed protection works is continuing. From the data in the bathymetric surveys (see Annex B) it can be concluded that for several cross sections in the last two years the erosion rate is even more than 15 m. per year.

5.5 Evaluation and comment

Evaluating the present bank protection measures near the Meghna Bridge some remarkable features can be noted.

- Design approach

- The design studies by the Japanese were mainly focused on designing the Bridge itself. The river engineering aspects were treated as "ancillary works". Through the study however, it was discovered that this might be an underestimation of reality and it was decided to lengthen the Bridge a bit for safety reasons and to conduct further studies for bank protection measures.
- The study for bank protection measures has always been conducted with a "trial and error" method.

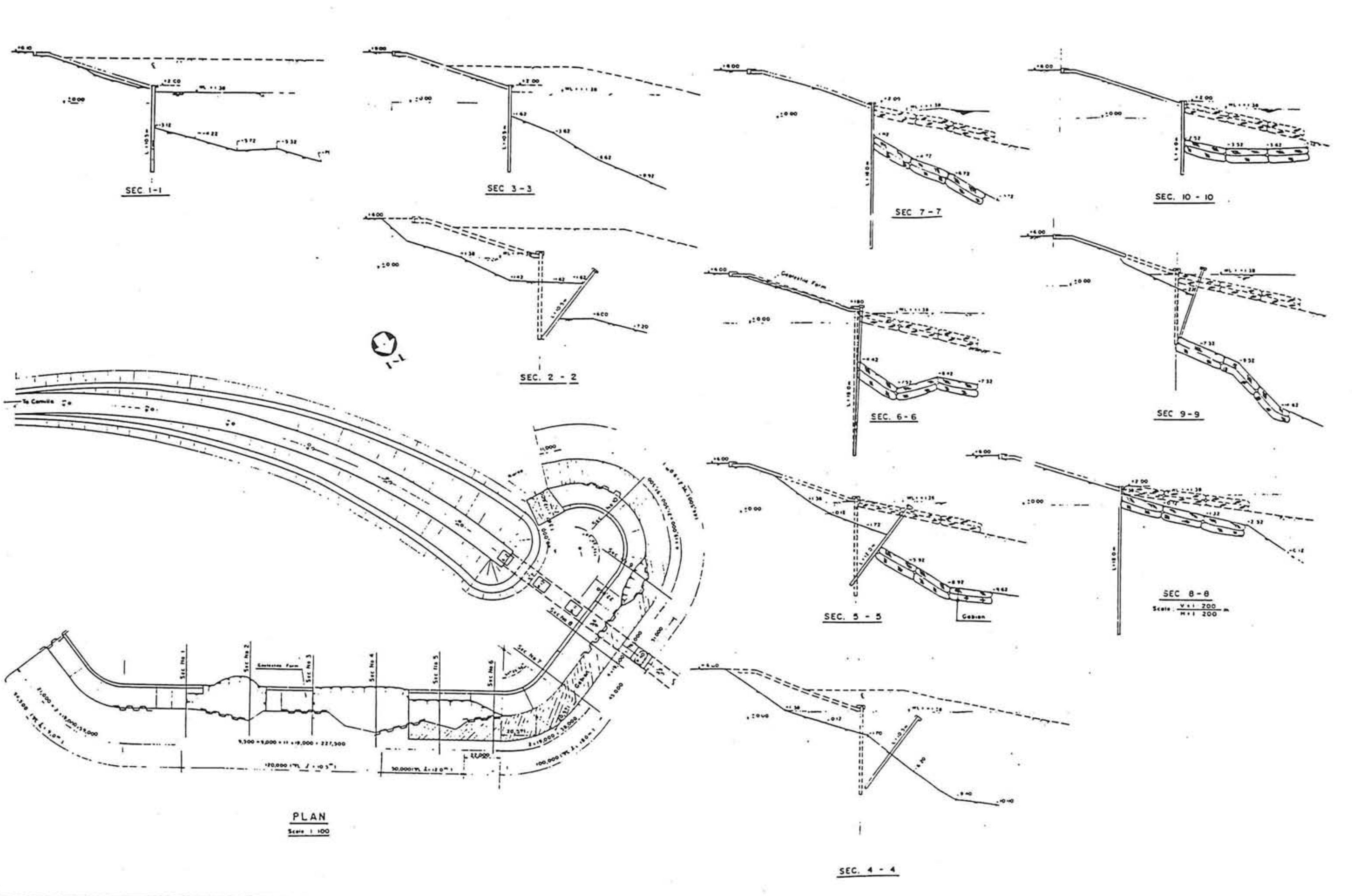


Figure I.7: State of bank protection works near the Meghna Bridge, June 1992 (source [7]).

- Materials used

- In the first design, soil filled jute bags were planned as river bank revetment. Jute however, will have been rotten within a few months. The type of soil is not specified but hopefully it was sand because clay can be flushed away through the pores of a jute bag.
- The bank protection, as constructed, is extremely rigid (non flexible). It is not able to adapt to small deformations of the subsoil or the river bed. This has become painfully clear in October 1991, when the revetments collapsed.
- Besides the fact that the materials used are not the most preferable for these purposes they are also not from local origin. The steel sheet piles used in the bank protection works are imported from Japan.

- Alignments chosen

- The alignment of the originally planned bank revetments, before the slope failure occurred, was quite smooth to the river direction. However, the strength of this construction, as mentioned, would not have been satisfactory.
- The alignment of the finally constructed bank protection is not very smooth to the river direction. In fact it is a revetment of the approach road embankment. Therefore the attack from the current is quite heavy.

It is also remarked that the piers for the two extra approach spans are only reaching down to 38.00 m-PWD, while the piers in the middle of the river bed are reaching to 54.00 m-PWD. Therefore, if a newly foreseen bank line would be located more in the vicinity of the abutment this can cause scouring problems around these piers.

5.6 Conclusion for this study

From the original design two important things can be learned and used for this study:

- Contrary to the "existing" bank protection the bank protection designed in this study has to be more of a flexible nature in the sense that small deformations of the sub base can be easily coped with.
- The alignment of the structure plays a major role, not only for its effectiveness but also for the loads on the structure. Unfortunately it is not feasible to perform scale model studies for this study. Therefore when designing alignments for bank protection works use has to be made of general experience (for instance from the S.T.S. and the Jamuna Bridge Studies).

Therefore within the design study (part II) durability and flexibility of the structures will be starting points.

PART II: DESIGN STUDY

Introduction:

In this part of the study designs, which solve the problems as defined in part I of this study, are derived and detailed. Finally a choice is made by means of a Multi Criteria Evaluation which results in a finally recommended solution.

In order to give a good insight in the way the design is created the entire design study, and not only the final result, is presented here.

The design study is composed as follows:

- Firstly, in Chapter 6, the most promising layouts which can fulfil the demands, are determined.
- Subsequently for the structures foreseen in these layouts the most promising construction materials are determined (Chapter 7), which finally will result into three overall alternatives.
- In the next Chapter these three alternatives are dimensioned and detailed (Chapter 8).
- Finally they are evaluated by means of a Multi Criteria Evaluation and the best solution is determined (Chapter 9).

6 ALTERNATIVE DESIGN LAYOUTS

6.1 General considerations

6.1.1 Introduction

In this Chapter the design study starts with a discussion concerning the possible solution types and layouts to solve the problems at the location of the Meghna Bridge. Finally the most promising layouts are determined which will be dimensioned and evaluated in the further study. Here possible construction materials are not yet discussed, this will be done in the next Chapter.

6.1.2 Requirements to be satisfied by the solutions

From Section 1.3.2 in part I of this Report, where the problem setting for this study is presented, it is known that the problems at the location of the Meghna Bridge can be divided into two parts. Therefore solutions must fulfil the following requirements:

i. The left abutment of the Bridge must be protected.
ii. The Bridge must keep its function i.e. be prevented from outflanking.

The second requirement is quite a vague statement. Therefore in Section 6.2 it is discussed what are the implications of this requirement and to what extent the present river layout has to be fixed or its banks must be protected.

6.1.3 Design approach

As already stated before the final design has to serve two purposes. Therefore the following approach is used for the determination of the most promising design layouts:

- First solutions are discussed to protect the left abutment of the Meghna Bridge.
- In a second step solutions are discussed to prevent the bend just upstream from the Bridge from outflanking and caving.

Combining the alternatives generated in these two steps results in overall solutions.

It was remarked already, in Section 1.3.2 in part I of this Report, that solutions for both these problems might (but not necessarily need to) be combined ones. However, an "overall at once" approach, which would generate solutions for both problems at once, is not elaborated here. Such an approach would lead very easily to oversized solutions which don't take the particular characteristics of the sub-problems in account. It is regarded to be a better approach here to compose the overall solutions from the best sub-solutions.

After generating and discussing the various possible solutions for the two sub-problems an evaluation is done and a preliminary conclusion concerning the most promising solution types is drawn. These solution types are discussed more extensively and their layout is determined more exactly in the Sections afterwards.

6.1.4 Design considerations

The alternatives, as they will be elaborated here, will be worked out keeping the following considerations in mind:

- Within this study it is assumed that the present general layout of the Dhaka-Chittagong highway will not be changed around the location of the Meghna Bridge within the first decades. This means that at the Bridge location also a possible new Bridge is projected if it is revealed that the present Bridge is too small or does not satisfy for any other reason.

- The design will be based on the data available at this moment and which are analyzed and presented in part I of this report and the various Annexes.

- The basis for the design is the layout as given in Annex D, Figure D.4. This layout is assumed to be a characteristic meander layout which is reasonably stable (see Annex D). The used bathymetry is from the survey of June 1992 and is presented in Annex B.

- The designed structures must consist of local (Bangladesh) materials as much as possible. Also the equipment and labour required have to be in accordance with the local experiences and skills as much as possible. This however, will be discussed in more detail in Chapter 7.

- It must be possible to construct the designed structure(s) within one dry season. In the next flooding season the structure(s) must be able to resist the loads fully. This has its impact on the required construction methods and the possible dimensions of structures.

- The proposed solution must be sufficient for a minimum period of about 50 years. This study is not only focused on short term emergency measures.

6.1.5 Criteria

The criteria to evaluate the various alternatives (both for abutment protection and outflanking prevention) are:

- Guidance of flow

This means that the alignment has to fit well within the stable planform as assumed and that the structure(s) guides the river flow smoothly through the Bridge. In such a situation the fewest eddies and vortexes will occur. Eddies and vortexes increase the risk for erosion and scour and are therefore not desirable. A stabilized and well guided flow will decrease the risk for problems in the future.

- Functionality

The proposed alignment should fulfil the demands. At the crossing with the Bridge the river must be fixed, and the Bridge must be prevented from outflanking by the river. If a structure is proposed it must be able to withstand the loads it is going to be exposed to. Further it is required that the solution is stabilising the situation for a period of about 50 years (long term solution).

- Practicability

It must be possible to construct the proposed structure within the given construction window of one dry season. Also it is preferred to use local labour and equipment. To be able to construct the protection works with local labour it is preferred that these do not require "high-tech" equipment, skills or construction techniques. Here the construction aspects are only regarded in general.

- **Costs**

As far as the costs are concerned only a first estimate of the construction costs is taken into account to compare the various alternatives for the construction materials (Chapter 7). When the final alternatives are elaborated (Chapter 8) also preliminary cost estimates will be elaborated which are used for the final evaluation (Chapter 9).

6.2 River scheme to be protected

One of the biggest questions within a river training study is always the determination of the reach which needs to be studied, this question is discussed in this Section.
Here the scales and sizes of the required river works are discussed. First the aspect of outflanking is discussed and subsequently it is determined which river stretch will be concerned with in this study. Within this discussion it is assumed that the Bridge itself is safeguarded already and that here it is only the requirement to keep the river flowing through the Bridge.

6.2.1 Outflanking

When taken literally outflanking itself is not the biggest threat for the location of the Meghna Bridge. In the S.T.S. the erosion rate is estimated to be around 15 m a year and therefore a considerable amount of time is needed to endanger the approach roads (which then will have to be revetted) or to develop a situation where a cut off of the channel through the Meghna Bridge could occur. Assuming that the flow is still kept guided through the Bridge (i.e. the Bridge abutment and the road embankments are protected sufficiently) the final situation could be as presented in Figure II.1.

It must be emphasized that this Figure is purely theoretical.

The bend characteristics, as used in this Figure, are taken from the data of the S.T.S. where the following observations were made:

"For the Upper Meghna in general the radii of curvature for the main channel vary between 1,250 and 12,500 m, while the arc length varies between 40 and 140 degrees. Derivations up to 110° from the valley direction occur. A clear correlation between the radius of the bends and the arc length of the bends was observed. A large radius of curvature results in (or may be better: allows only) a small arc length."

Therefore the occurrence of a derivation of 110° from the valley slope (i.e. a large arc length) in conjunction with a bend radius of 12,500 m., which leads to even a more critical situation (river even nearer the approach road embankments) is not very probable. However, it would not be very wise if, because of this reason, no attention would be paid to the aspect of outflanking.

The forecasted erosion pattern (caving into the left bank) leads to a situation where the approach flow to the Bridge is becoming more and more oblique. For such an oblique flow the constriction rate at the Bridge increases and just upstream a bend, with a very small radius, will develop.

This induces deeper scours and heavier attacks on the Bridge's abutments and piers (through time also

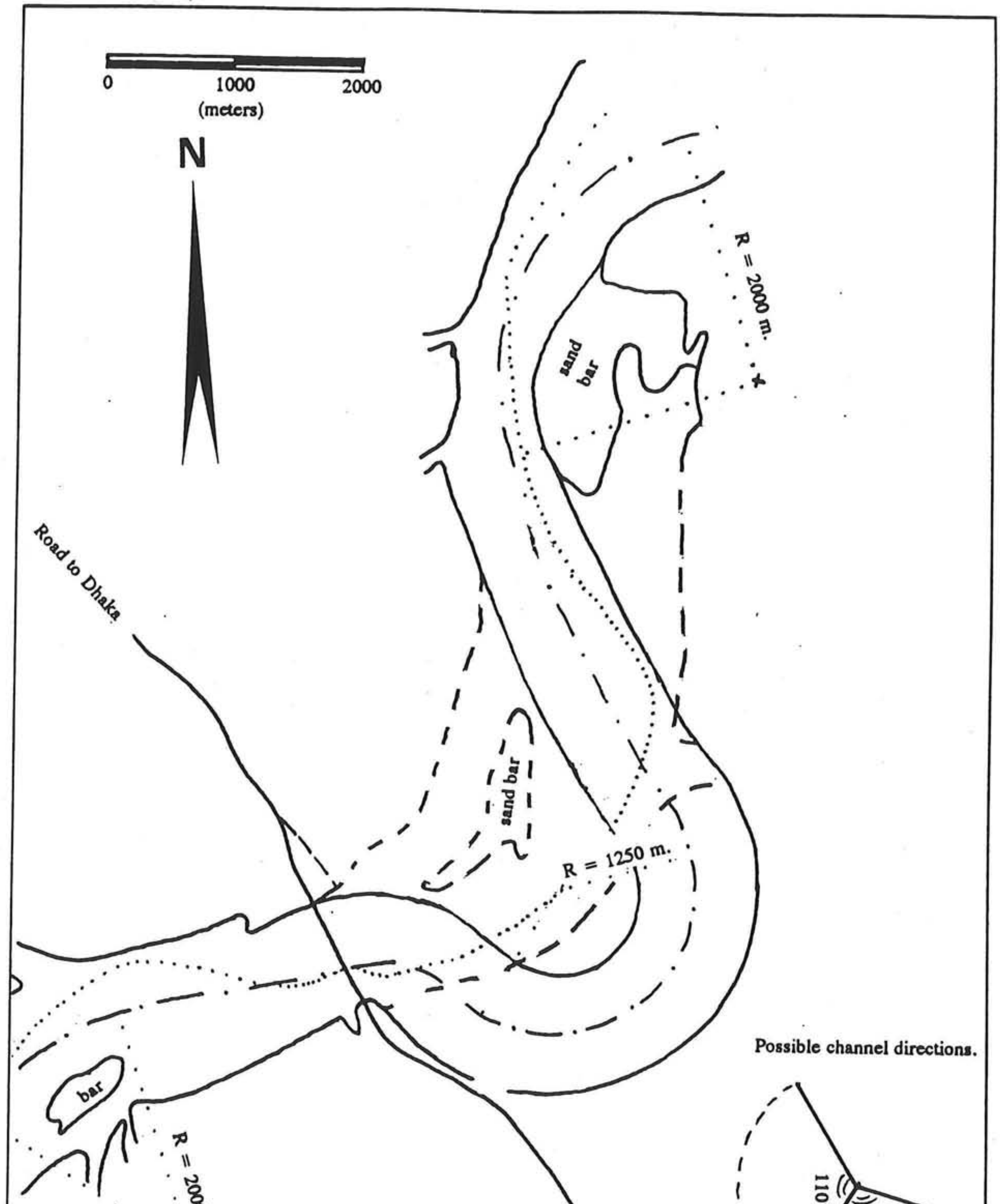


Figure II.1: Final situation for worst outflanking scenario.

the right abutment can be attacked). As such a situation would require further improvement of the Bridge's abutments (making them heavier) it is wise to take some measures now to prevent such a development.

6.2.2 Length to be protected

From the bathymetrical surveys, as presented in Annex B, it can be seen that in the stretch upstream from the Bridge reaching to cross section U3, the erosion has been quite severe during the last few years. This is not so strange as just upstream from cross section U3 the lower end of the sand bar is located and therefore at this point the constriction rate is the highest. In cross section U4 however, the erosion has been quite small. This means that the river bend is caving itself into the left bank between cross section U4 and the Meghna Bridge.

Therefore it is decided that within this project a study will be made to measures to fix the left river bank up to cross section U4. This is a reach of about 2700 m starting upstream from the former Ferry Ghat (cross section U1).

Unfortunately no more specific data are available of the left bank at cross section U4. It is not known whether the erosion rate is so low here because the hydraulic loads on the river bank are low or that the bank is more resistant (for instance due to the presence of clay here). Most probably it is a combination.
The two dimensional flow velocity model, as elaborated by the Japanese consultants and presented in Annex C (figure C.1), reveals a drop in the flow velocity along the left bank at cross section U4. More upstream from this cross section the flow velocities are higher along the right bank. Cross section U4 therefore can be considered as a transition point in the meander. At cross section U4 the Thalweg is situated very near to the left bank which results in a steep slope. However, compared to other cross sections, this slope is not under heavy erosion. This characteristic could indicate also the presence of clay here.

Bank protection at the concave side of the eroding river bend (see Figure I.2) will not immediately stop the shifting of the sand bar in the direction of this bank. Here (with the river bed being "too wide") it can be expected that accretion along the bank proceeds independently of what happens along the opposite bank. Fixation of the left bank will cause the river to deepen, as the convex bar will continue to shift to the left bank. At a certain moment however, the constriction will provide a current strong enough to prevent further shifting of the sand bar in the inner bend which however, will continue to gain in height.

6.3 Alternatives

Here the alternative solutions to fulfil the requirements, as listed in Section 6.1.2 are discussed. The discussion is split into the two parts which were mentioned already when discussing the design approach.

6.3.1 Protection of the left Bridge abutment

For this particular problem the zero option (doing nothing) is not mentioned as it is known from the problem setting that this is a non tolerable option.
The protection of the left abutment can be achieved in various manners. A distinction can be made

between strengthening or smoothening. Alternatives are:

1. **Constructing a new left Bridge abutment** (at the same location).
   This can be done by extending the abutment "plateau" at 6.00 m+PWD and constructing a new (and less steep) upper and underwater slope and a reconsideration of the revetment materials to be used.

2. **Lengthening the Bridge.**
   Already during the Bridge construction its length was extended by two spans of 25 m each. This can be done further. This would lead (temporarily) to a decrease of the dangers induced by bank erosion and safeguarding of the (new) Bridge abutment.

3. **Construction of a guide bank.**
   Instead of solely improving the design of the present abutment (strengthening) it is also possible to design a more extended abutment shape which also guides the flow under the Bridge. In this way the protrusive character of the present abutment can be decreased. The shape of a guide bank is mainly influenced by the principle of a "guide bund", which is a common hydraulic structure in the Indian subcontinent.

The fulfilment of the various alternatives to the criteria can be summarized in a score card:

***Table II.1:** Comparison of alternatives for abutment protection.*

Criterion / Alternative	Guidance of flow	Functionality	Practicability	Costs
Constructing a new abutment	-	o	+	A
Lengthening the Bridge	-	-	+	A+
Construction of a guide bank	+	+	+	H

The marks in the comparison Table signify the following:

- \+ no problems or problems of a limited (acceptable) nature expected,
- o certain problems expected which are solvable but require special attention,
- \- serious problems expected,
- A average (cost),
- A+ above average (cost),
- H high (cost).

The scores to the criteria can be explained as follows:

- Concerning the **guidance of flow** only the construction of a guide bank is a good solution, here the present vortex area, between the present left Bridge abutment and the former ferry ghat, can be eliminated. The other two solutions do not contribute anything to this matter.
- Also due to the remarks made above the **functionality** of the alternatives other than the construction of a guide bank, is rather cumbersome. Constructing a new left abutment will result in a heavy structure which is still heavily attacked. When lengthening the Bridge the crossing of the Bridge and the river is not fixed at all, new problems will come up later.
- For all three solutions the **practicability** is expected to be sufficient. Therefore here no distinction is made. For the alternative of lengthening the Bridge it is assumed that use can be made of the equipment present in Bangladesh at the moment for the construction of the

Meghna-Gumti bridge (the bridge over the second channel of the Upper Megha River here) which is located at a distance of about 12 km.

- The construction **costs** for the various alternatives vary with the sizes of the proposed structures. Constructing a new left Bridge abutment is expected to be the least expensive as this is only an addition to an existing structure. Lengthening the Bridge involves some more work and the construction of a guide bank is expected to be the most expensive. It must be borne in mind that here only the construction costs are considered. When the Bridge is extended also extra costs due to the interruption of traffic etc. will arise.

Some remarks concerning the alternatives must be made:

The alternative of **improving the existing abutment** does not change anything to the flow pattern just upstream which brings along a heavy attack on the abutment. Therefore this option is not desirable.

Increasing the Bridge length is not a very good solution. When implementing this the river still is not guided or fixed at the crossing with the Bridge and the erosion will still continue. The number of spans required is very difficult to determine and the navigational clearance is not maintained as the navigational channel will also shift further to the left bank where the Bridge height will be less. Another complicating factor here is that the piers 10, 11 and 12 (for their location see e.g. Figure I.6) are founded less deep in the river bed and therefore might become vulnerable to scour when the thalweg shifts further towards the left bank.

The **construction of a guide bank** seems to be the best here as it combines two important aspects: the abutment is stabilized and the river flow is guided more smoothly through the Bridge. The flow pattern near the left abutment will be less turbulent as the vortex area will disappear. Therefore it is expected that when this alternative is implemented the scour rates will decrease as well.

6.3.2 Prevention to outflanking

As mentioned already to prevent the Bridge from outflanking by the river and stabilize the situation here on the long term the bend upstream more or less has to be fixed in its present position.
A distinction can be made between (re)direction of the river flow and measures to strengthen the present river banks. Also mixtures of these approaches can be recognised. Here also the so called "zero option" is not discussed. In fact this is done already when dealing with the question whether outflanking has to be prevented or not (Section 6.2.1). The alternatives are as follows:

- Bank fixation measures

(1) **River bank revetment (present banks)**
A revetment having a total length of about 3000 m is needed. Though the exact soil characteristics at cross section U4 are not known it is assumed here that, for a proper design, bank revetment must be extended some distance into the wide side channel just upstream from cross section U4. The stretch into this channel is taken equivalent to the width of this channel being 300 m.

(2) **River bank revetment (advanced)**[4]
This is the same solution as outlined in (1) but now the river bank is "trimmed". The small channel just downstream from cross section U2 is closed and at some points the present bank

[4] Here "advanced" does only refer to the position of the bank revetment but not the used techniques.

is shifted forward a little.

- Mixtures of bank fixation and flow (re)direction

(3) **Series of groynes.**
A series of groynes can be constructed to fix the present bend layout and guide the river flow through it. The number of groynes depends on the applied length.

(4) **Series of hard points.**
The points at the river bank where at present the erosion rate is high can be reinforced thereby creating so called "hard points".

- Flow (re)direction

(5) **Bandalling.**
The system of bandalling (i.e. the construction of bottom or surface panels) could be implemented. When constructing a set of bottom panels in the flow area, the flow perhaps can be redirected from the banks and to the Bridge.

(6) **Dredging.**
The inner bend channel, going right around the sand bar, in the bend upstream can be dredged. This leads to a decrease of the flow in the outer bend channel which is the main cause of erosion inducing the danger for outflanking. Dredging the convex side of the outer bend (the sand bar) is of no use as this will not influence the flow pattern at the concave side.

(7) **Decreasing the discharge through the Meghna Bridge channel.**
If the discharge through the Meghna Bridge channel decreases also the erosion rate will decrease and thereby the danger for outflanking. This could be achieved by constructing a groyne at the upstream bifurcation which guides the flow more to the Meghna Gumti channel.

The fulfilment of the various alternatives as mentioned above to the criteria can be presented in a score card:

Table II.2: *Comparison of alternatives for outflanking prevention.*

Criterion / Alternative	Guidance of flow	Functionality	Practicability	Costs
1) River bank revetment	+	+	o	A+
2) Advanced river bank revetment	+	+	o	A+
3) Groynes	+	+	o	A+
4) Hard points	o	o	+	A
5) Bandalling	+	-	-	A-
6) Dredging	o	o	o	A
7) Groyne at bifurcation (discharge decrease)	o	-	+	A

The marks in the comparison Table signify the following:

+	no problems or problems of a limited (acceptable) nature expected,
o	certain problems expected which are solvable but require special attention,
-	serious problems expected,
A-	below average (cost),
A	average (cost),
A+	above average (cost).

The score to the criteria of the various alternatives can be explained as follows:

- **Guidance of flow** is considered for the proposed structure to the planform layout. The guidance is the best for the first three alternatives, river bank revetment fixes the river bank which is shaped by the flow and therefore the guidance is quite smooth. This guidance of flow can be expected to be even more smooth when an advanced revetment is applied and sudden kinks and gaps are eliminated. Also when groynes are applied the guidance of flow will be satisfactory, though they are orientated perpendicular to the flow their guidance is still quite smooth. Hard points will not guide or direct the flow very well, they are more like obstructions. Bandalls can guide the flow quite smoothly. The option of dredging leads only to a temporary flow guidance, after a while the flow will cave its own way again. The last option doesn't guide any flow in the vicinity of the Bridge.
- The **functionality** of the various options is quite varying. From the first three, almost classical, methods it is known that their functionality will be satisfactory, also for the circumstances at the Meghna Bridge. Hard points will be quite vulnerable to outflanking themselves and therefore their functionality is doubtful. Bandalling is an option which functions well for situations of a smaller scale. However, in this situation this option cannot be expected to be functional. Dredging is functional only for a short period. This option will induce a new problem however as through time two oblique channels with a wig in between will develop. The functionality of a groyne at the bifurcation is not sufficient as this will shift the problems to the other channel without solving anything.
- Especially for the option of bandalling the **practicability** is very low, it will not be very easy to construct panel structures in the river bed and to keep them in position. Also for the larger structures as revetments and groynes the practicability will cause some problems, here however it is expected that these will be solvable.
- The **costs** are relative to the sizes and scales of the proposed structures.

Some options need some additional comment:

Bandalling is a solution which is not practical for this particular situation. The construction of bottom panels needs to be done in the bends where the depths even in the dry season are about 15 m. Also the bathymetry here is continuously changing. Bandalling is a solution type which is not applicable in these "large scale" situations.

When **dredging** the inner bend the characteristic layout, as determined in Annex D and presented in Figure D.4, is disturbed to a high degree. Therefore it is expected that such a newly created situation will not be stable. The shifting velocity of the sand bar will increase. Besides the expected sedimentation in the dredged trench also the meander course can change. Though the development of such a change needs a lot of time it is not preferable at all to induce it. Another disadvantage of this solution is that the newly created channel will not have a perpendicular approach to the Bridge either. In between the sand bar will shift downstream to (or under) the Bridge and therefore the constriction at the Bridge will increase and the loads on the left abutment will increase as well.

The option of **decreasing the discharge** by redirecting the flow through the Meghna Gumti channel at the bifurcation is more an academical one. Besides the fact that it is almost impossible to derive to what extent such a redirection is needed it also redirects the problems to the other channel. The Meghna Gumti Bridge, which is presently under construction, is designed for the present situation and therefore problems can be expected there when this alternative would be implemented.

From the discussion above it can be derived that the (classical) options of revetting the river bank or the construction of groynes here are the most promising ones.

6.3.3 Preliminary conclusion

From the comments in Section 6.3.1 and 6.3.2 it can be concluded that for solving the problems at the location of the Meghna Bridge the following options seem promising:

- To protect the left abutment of the Bridge a guide bank solutions seems to be the most appropriate. An advantage of this solution is that it stabilizes and guides the flow through the Bridge.
- To stop the caving of the left bank upstream from the Bridge two options are promising: application of groynes and revetting the river bank. As no clear distinction of the merits and disadvantages of these options can be recognised yet, both these options will be studied further.

Layouts of these most promising solutions are elaborated in the following Sections.

6.4 Guide bank layout

6.4.1 Introduction

The layout for the guide bank here will be derived taking some considerations for the design of guide bunds as a starting point. As for the design of most hydraulic structures, also for the design of guide bunds no standard recipe is available. However a general practise concerning this matter can be derived from handbooks (ref [14]) and the Jamuna Bridge Studies (ref [16]). In both it is also stated that for a proper design scale modelling is necessary. This however, is not feasible within this thesis project. Therefore the general practice is studied in Section 6.4.2 and adjustments are made to it for the location of the Meghna Bridge from the considerations as presented in Section 6.4.3.

6.4.2 General practice concerning guide bund design

The design of guide bunds as it is discussed here is derived from two sources namely the manual by Joglekar (ref [14]) and the Jamuna Bridge Studies (ref [16]). In the manual by Joglekar mainly is referred to the methods of R. Gales (1938) and F.E.J. Spring (1903). Here a description of the general practise for the determination of such a layout is presented.

Generally guide bunds are designed in pairs. In some situations however, it may occur that the construction of one guide bund only is sufficient. Here the general practice i.e. the design for two guide bund structures is discussed. The task of this pair of guide bunds is to guide the river flow through the constricted area which is caused by the construction of road approaches on embankments

in the main river bed.
Within this general practise it is also assumed that the river has a certain kind of flood embankment.

- Length upstream of the approach road embankment

This length is derived from two criteria: the first one deals with oblique flow attack (on the guide banks) and the second with the so called "worst probable embayment" which could endanger the approach road embankments.

- To prevent oblique flow attack on the guide bund structures it is attempted to guide the flow axially through the Bridge. The most dangerous condition is when the main stream first attacks the head of one guide bund then swinging onto the other near the Bridge abutment, to attack the foundations of the adjacent piers. A guide bund with a length upstream of about 1.00 to 1.25 times the Bridge length will be sufficient to avoid such a situation.
- The "worst probable embayment" holds the following: If a guide bund is constructed in the river bed, the river can start to outflank this construction. However, this outflanking is limited. The "worst probable embayment" (see Figure II.2) is the limit state of this outflanking channel. This limit state is derived from fitting the bend with the smallest radius, derived from the river reach upstream of the guide bund location, to the head of the guide bund. This bend is fitted between the top of the guide bund and the flood embankment. So the safety margin between approach road embankment and outflanking channel is reviewed. The length of the guide bund upstream from the approach bank can then be chosen so as to leave a substantial margin between this worst probable embayment and the road approach bank.

- Length downstream of the approach road bank

The downstream reach is determined by the demand to provide a smooth outflow preventing the Bridge from being endangered. The downstream length of a guide bund is usually 0.10 to 0.25 times the Bridge length. With these basic design rules a first design is elaborated. Next with scale model studies it is checked whether this design can withstand the loads when the "worst probable loop" upstream is established.

In the manuals it is remarked that, if further protection is needed, additional training can be established by way of groynes or revetment for the road approach banks. In the manual from Joglekar it is advised not to construct groynes or hard points in the upstream river stretch but along the approach road embankment(s).

Field data from India revealed that this design method provides satisfying results as no calamities occur.

6.4.3 Considerations for the location of the Meghna Bridge

When comparing the situation at the location of the Meghna Bridge to a general situation as presented in Figure II.2 (next page) it becomes clear that the general practise is not directly applicable to this location. Normally, with a flood embankment, the meandering/outflanking pattern is free within the limits of the flood embankment. In this particular situation however, no flood embankment is existent and outflanking/meandering must be limited by means of river bank protection upstream of the Bridge, see Section 6.2.2.

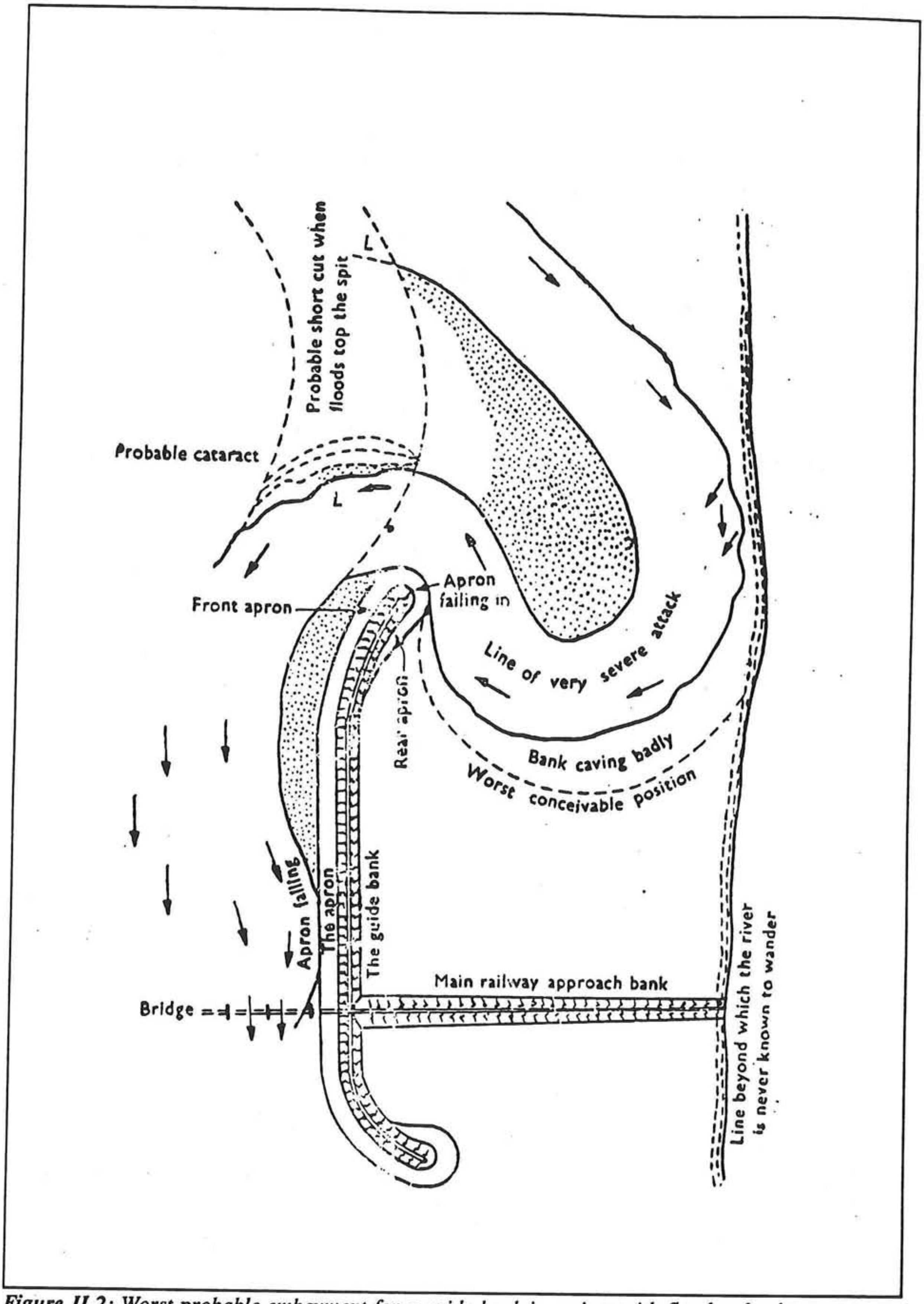


Figure II.2: Worst probable embayment for a guide bank in a river with flood embankments.

Therefore to the guide bund design practice the following adjustments are applied:

- With the location of the Meghna Bridge being at a bend only the left bank is threatened. Therefore the construction of only one guide bund at this side of the river must be sufficient.
- As only one guide bund is constructed its upstream length is not determined by the demand of preventing oblique attack with the rule of thumb for a pair of guide bunds. Oblique attack here has to be prevented by a smooth alignment of the guide bund to the general direction of the river axis.
- Derivation of the worst probable embayment does not hold here. Outflanking is prevented by bank protection at the concave side of the bend upstream.

It must be clear that the proposed solution is called a guide bank and not a guide bund.

6.4.4 Determined guide bank layout

- Length

Upstream limit

The upstream limit of the guide bank is governed by practical considerations. To stabilize the flow pattern the guide bank must close the present vortex area. Thereby use can be made of the former ferry ghat which is already acting as a strong point. Taking this former ferry ghat as the upstream head of the guide bank its upstream length is determined. Lengthening the upstream part of the guide bank would not serve any particular purpose and therefore is not done.

Downstream limit

There are several possibilities for the downstream end of the guide bank. These are the following:

i. The guide bank can be constructed as one continuous construction reaching down to the downstream end of the Bridge abutment, thereby also improving the Bridge abutment.
ii. It is also possible however, to locate the downstream end of the guide bank just upstream of the Bridge abutment thereby leaving a gap in between this guide bank and the abutment in such way that the little bay, which is created by the slope failure in 1989 (see part I, Section 3.5), still can be used as a little harbour. The improvement of the Bridge abutment then has to be treated as a separate topic.

The little harbour, as it is used at the moment, is only used by the Bridge contractor for the emergency works. When the gap would be maintained, it can induce turbulence in the river flow here which will be an extra load on the abutment which is not preferable. Also the value of a (very little) harbour here is not clear and therefore the possibility, as presented under i., will be implemented. Depending on the material used to construct the guide bank (see next Chapter) this small port could be reclaimed or not.

N.B. If ever desired, a small harbour can also be constructed downstream of the left abutment.

For the length of the guide bank downstream of the Bridge only a small extension is foreseen compared to the present Bridge abutment. The downstream side of the abutment is not damaged and the erosion pattern here is not very severe. Therefore there is no reason to lengthen this downstream end for the new situation. The small lengthening compared to the present abutment is only foreseen to create a smooth guide bank layout which fits smoothly around the present abutment of the Bridge.

- Guide bank orientation

In theory the orientation of the guide bank is optimal when it is directed along the river axis thereby providing a smooth flow pattern.
For the orientation of the guide bank in fact two options are available.

i. The direction can be optimized by guiding the river flow as smoothly as possible, this would result in a situation with the head of the guide bank located through the remainders of the former ferry ghat. This ferry ghat partly needs to be destructed then.
ii. Another possibility is to construct a guide bank with its head being the remainders of the former ferry ghat which would be supplied with a proper revetment.

Evaluating this is a more subjective matter. Both solutions have their advantages. The first solution results in an optimum guided flow while the second possibility takes advantage of the relatively higher strength of the remainders and protection works of the former ferry ghat compared to the other bank parts.

However, the former ferry ghat is still under erosion and it is also possible that this problem solves itself. By the time that the construction can be started the former ferry ghat may be eroded so much that the orientation of the second option, as presented above, is equal to the orientation of the first one.

As the difference between the two options is small and even decreases with time for this study the second possibility is chosen because of its construction profits.

- Curved head and end of the guide bank

To establish a smooth guidance of the flow, it is preferable that the guide bank is designed without sudden kinks where a flow can "drop off". Avoiding these "drop offs" also avoids the creation of eddies which can induce scour. Therefore the head and the end of the guide bank are curved. At the head this curve is composed with radii of 80 and 120 m respectively. In this way it fits easily around the former ferry ghat.

To avoid a sudden kink at the place were the guide bank reaches the abutment a slight curve is introduced here also. This curve passes into another curve with a smaller radius. Here the radii are 180 and 40 m respectively.

A schematic picture of the layout, as determined above, is given in Figure II.3. This figure must be regarded as a rough presentation of the idea. No details are given concerning the type of materials, slopes, toe protection etcetera. These will be determined later.

As a first estimate a crest width of 6.0 m. is applied. This is a fairly practical value. In a later stage this crest width can be reviewed however.

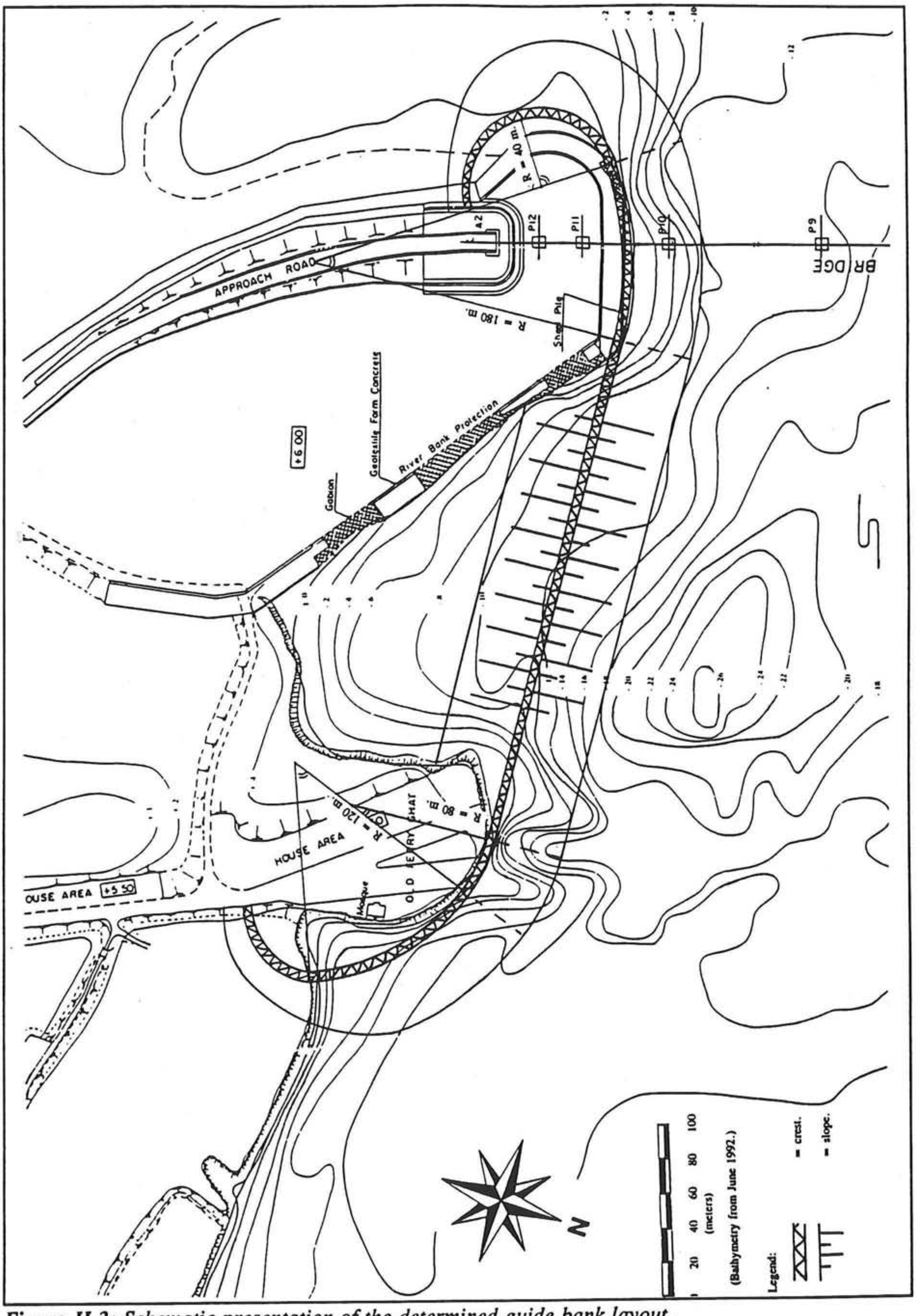


Figure II.3: Schematic presentation of the determined guide bank layout.

6.5 Layout of bank protection works

As there is more than one option left here this Section is split into two parts, first the option of river bank revetment will be discussed and afterwards groynes are discussed.

6.5.1 River bank revetment

When revetting the river bank it is chosen to apply a so-called advanced protection. Here advanced (as opposed to "retired") only refers to the position of the protected bank and not to the used techniques. The protection is chosen to be advanced because in this way gaps and kinks can be filled out thereby creating a smooth alignment/guidance of flow. At places where the river bank alignment is smooth already at the present bank only an protected slope has to be created. At other places the position is a little advanced to the present bank line. The small side channel, just downstream of cross section U2 is closed to prevent any outflanking pattern to develop. The discharge through this small channel is assumed to be of no importance.

The upstream end of the river bank revetment is determined at 300 meters into the side channel just upstream from cross-section U4. In this way a continuous protection is established which has a good resistance against the hydraulic loads. Ending the protection downstream of cross section U4 would result in a very vulnerable point of the structure at its upstream end.

6.5.2 Groynes

In the groyne alternative some small erosion between these groynes can be expected. This is inherent when a point to point guidance of the river is established.
Here no protection or revetment is foreseen at cross section U4. However, at a later stage, the protection length can be extended more upstream. But within the scope of this project this is not necessary.

- Length of the groynes

From the geomorphological data analysis it is known that the first bend upstream from the Bridge at present has a normal bend radius of approximately 2000 m. As the average bankfull width at the Bridge is around 900 m the radius of the outer bend is 2450 m. So to stabilize the bend with a radius of 2000 m. the optimal alignment for a series of groynes is to fit their heads to a circle with a radius of 2450 m. The best circle layout is presented in Figure II.4.

However, from Jansen (1979) it is known that for stable and trained river bends the bend radius may decrease within the bend but not increase. In this way it is be assured that the flow will be stabilized. This is shown in Figure II.5.

Therefore it must be possible to expand this small river training scheme in the future. For such a lengthening of the protected river scheme the best also an outer bend radius of 2450 m can be applied, in this case the radius downstream can be decreased to 2250 m thereby assuring the continuation of a stable flow pattern. The outer bend layout for possible future extensions is depicted in Figure II.6.

For such an extension projected groynes, upstream from cross section U3 would have to be lengthened a bit.

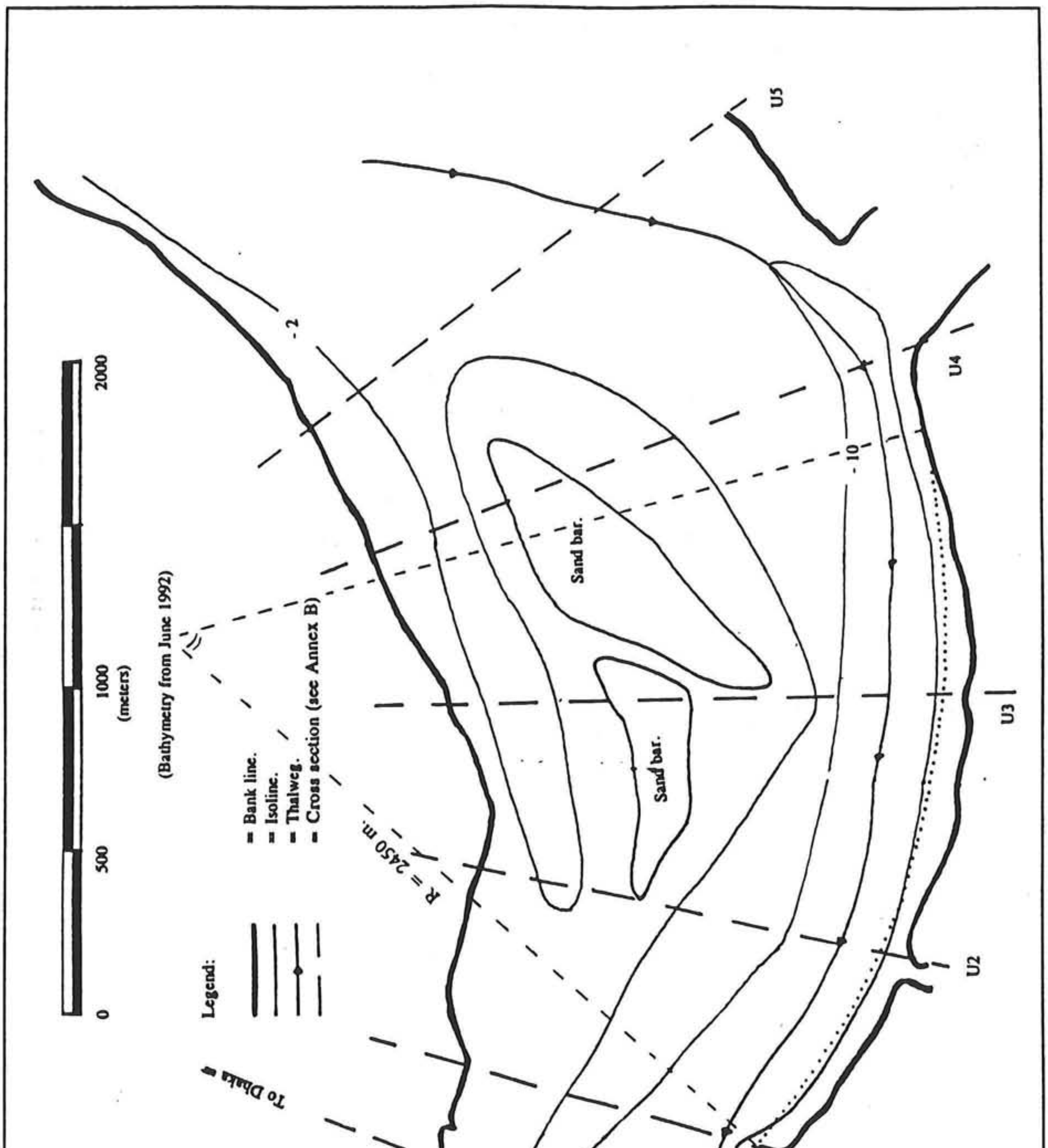


Figure II.4: Optimum outer bend alignment for groyne alternatives.

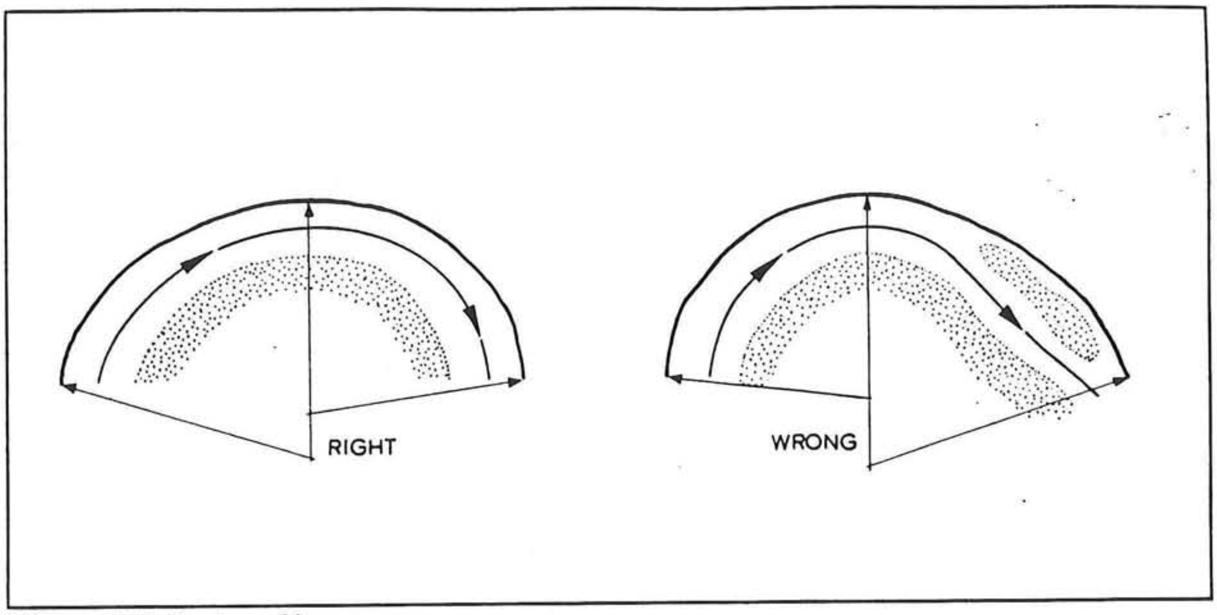


Figure II.5: Design of long curves.

It can be noted that in the situation as proposed above the approach flow to the Bridge is still not perpendicular to the Bridge axis. However, it is assumed that this will not cause major problems as the abutments and piers of the Bridge are designed to the present flow situation which is fixed here and it is assumed that these are stable enough.

- Applied groyne type

To prevent the groynes from inducing any outflanking pattern themselves the crest should not be taken higher than the hinterland and these should have a small slope towards the river. This holds also that the groynes are submerged during the periods when the hinterland is also inundated.

Groynes can be permeable or impermeable. A permeable groyne has the slight advantage of inducing less scour than an impermeable one. However, no models are available yet to quantify this aspect. Therefore here the choice between a permeable and an impermeable groyne is left to the discussion of the used construction materials but no preference is determined.

The orientation of the groynes to the thalweg, main flow direction or the river bank can also vary. Groynes can face downstream (so called attracting groynes) which results in an attraction of the flow in the direction of the bank or facing upstream (so called deflecting groynes) which concentrate the flow in the main channel. Also here however, no models are available to quantify this aspect. This could only be done in scale modelling studies which fall beyond the scope of this master thesis project. Therefore here the safest option is chosen which is an orientation perpendicular to the river bank and the projected curve from Figure II.4.

- Spacing

A spacing of 2 to 2.5 times their length is the general practice when groynes are applied for navigational purposes while 5 times their length is common practice for erosion protection in straight channels.

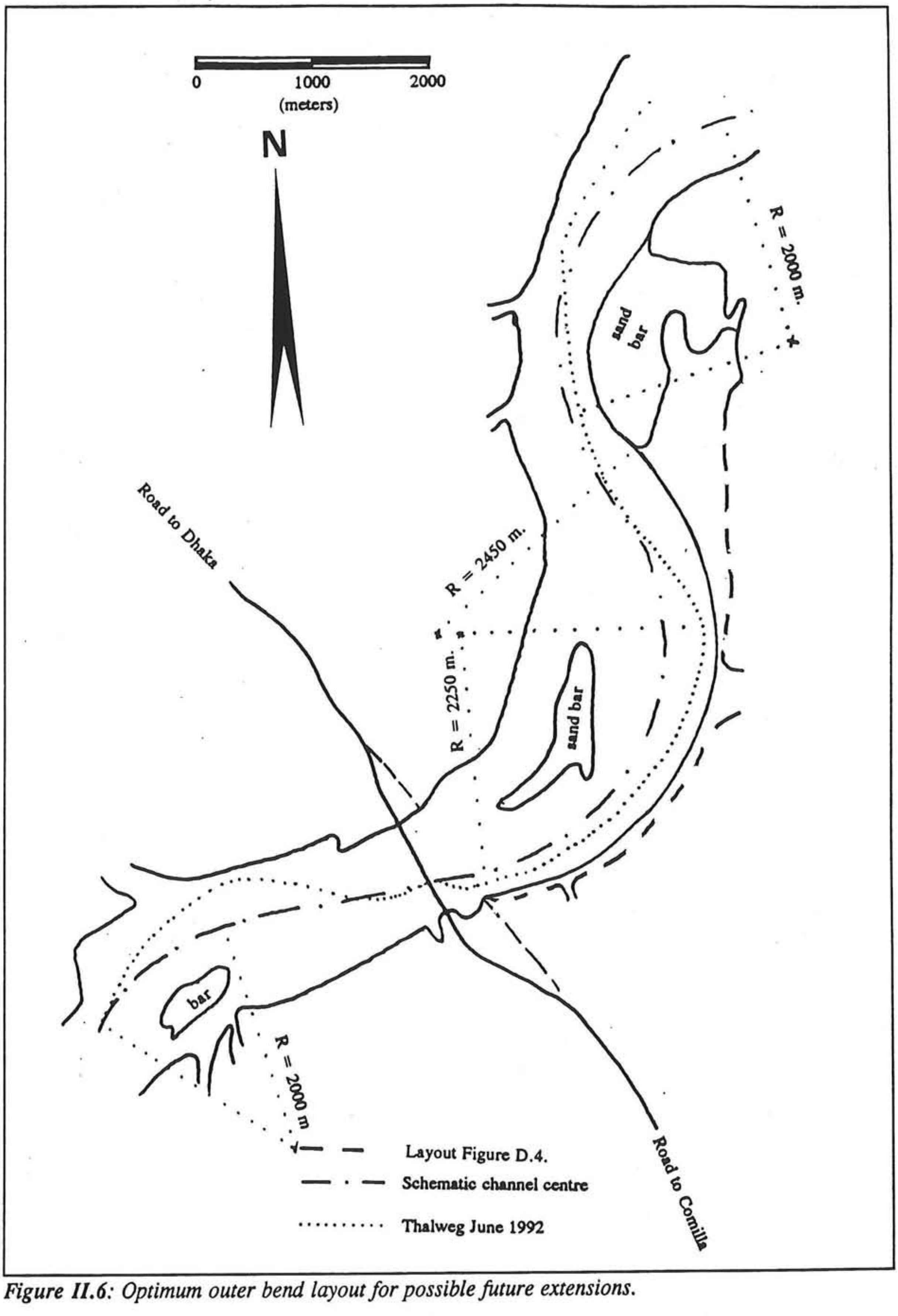


Figure II.6: Optimum outer bend layout for possible future extensions.

Sometimes, groynes are spaced far apart to lessen the cost of construction, or with a view to put more groynes in between at a later date, the result may be that either the flow is disturbed and the groynes outflanked, or their heavy maintenance costs exceed the saving attempted.

As the river bank has a curvature here and the groynes are foreseen in its concave bend a reduction must be introduced to the spacing. Assuming that the flow deviates in an angle of 1:5 from the head of the groyne and that the groyne, with a length of 100 m, is directed perpendicular to the general flow direction the point at which the flow hits the bank can be calculated[5] to be a little more than 300 m. Therefore a spacing of three times the length is adopted.

From the hydraulic theory the maximum spacing of a pair of groynes to secure stable eddies in between follows from:

$$L < \alpha \frac{C^2 h}{2g} \tag{1}$$

Where:

L	= space between the groynes (m),
α	= coefficient (-), this coefficient should be below unity, laboratory tests indicated that it should be at or below 0.6,
C	= Chezy factor ($m^{1/2}/s$),
h	= average depth between the groynes (m),
g	= acceleration of gravity (m/s^2).

This formula expresses the available head relative to the head required to establish a flow over a length L. Filling in $\alpha = 0.6$, $C = 50$ and $h = 5$ (all very conservative values) results in a maximum spacing of about 380 m.

One single elliptical eddy gives the best support. This requires a quotient of the axes in the ellipse in the x and y directions which is smaller than 2 (where x is the direction along the river axis while y is the direction perpendicular to x). This however is more a requirement for stable navigational channels. Therefore here no account is taken for this aspect.

When groynes are constructed in the present river bed with their heads fitted to the circle as depicted in Figure II.4, their length will be around 100 m. With a spacing of three times the length of the groynes this results in a series of 5 groynes with a length of 100 m and one groyne upstream with a length of about 60 m.

As the economic value of the hinterland is quite low it can also be considered to design groynes with a double length and therefore a wider spacing. As the head of the groynes is still fitted to the circle of Figure II.4 here part of the groynes is constructed in the hinterland. Still the spacing ratio is taken at 3. To provide a good eddy here a depth of about 8 m in between the groynes is required which is a reasonable value.

This alternative results in two groynes with a length of 200 m and one groyne upstream with a length of about 160 m. The spacing is taken three times the length of the groyne upstream.

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[5] This value follows from calculating the intersection of the equations describing the outer bend alignment and the line of the flow direction, deviating in an angle of 1 to 5 from the head of the groyne.

In this alternative a further (controlled) erosion in between the groynes of about 100 m. is anticipated. The erosion rate will be lower than the predicted 15 m per year for the present situation however.

Other (groyne) alternatives with a further lengthening at the land side are not considered as this would lead to undesired ratios of the river width to the groyne lengths.

6.5.3 Conclusion

For the river bend fixation three layout alternatives are left to elaborate further.

1) (Advanced) River bank revetment.

2) "River Training" by submersible groynes; two with a length of 200 m and one groyne of 160 m length. The spacing is taken three times the length of the groyne upstream. Part of these groynes is constructed in the hinterland.

3) Five, also submersible, groynes with a length of 100 m. and one groyne of about 60 m. Here the spacing is taken also three times the length of the groyne upstream.

6.6 Most promising overall design layouts

Finally here the total layout alternatives are determined which will be elaborated further within this study. After detailing these alternatives a final choice will be made by means of a Multi Criteria Evaluation (MCE) in Chapter 9.
From the discussion concerning the layout alternatives for abutment and river bank protection three overall layout alternatives can be determined. These are:

- Layout alternative I:

A guide bank to protect the left abutment of the Bridge and an advanced revetment with a total length of 3000 m. in the bend upstream. This layout alternative is presented in Figure II.7.

- Layout alternative II:

A guide bank to protect the left abutment of the Bridge and (long) groynes in the bend upstream. In total three groynes (LG1 to LG3) are foreseen; two (LG1 & LG2) with a length of about 200 m and one (LG3) with a length of about 160 m. As in this alternative further erosion of the river bank is foreseen also the upstream head of the projected guide bank (Figure II.3) must be a little extended. This can be done by revetting the embankment of the approach road to the former ferry ghat over a length of 200 m. This layout alternative is presented in Figure II.8.

- Layout alternative III:

A guide bank to protect the left abutment of the Bridge and a series of (short) groynes in the bend upstream. In total six (SG1 to SG6) groynes are foreseen; five (SG1 to SG5) with a length of about 100 m and one (SG6) with a length of about 60 m. This layout alternative is presented in Figure II.9.

For all types of structures in these layout alternatives the construction materials are discussed and evaluated in the next Chapter.

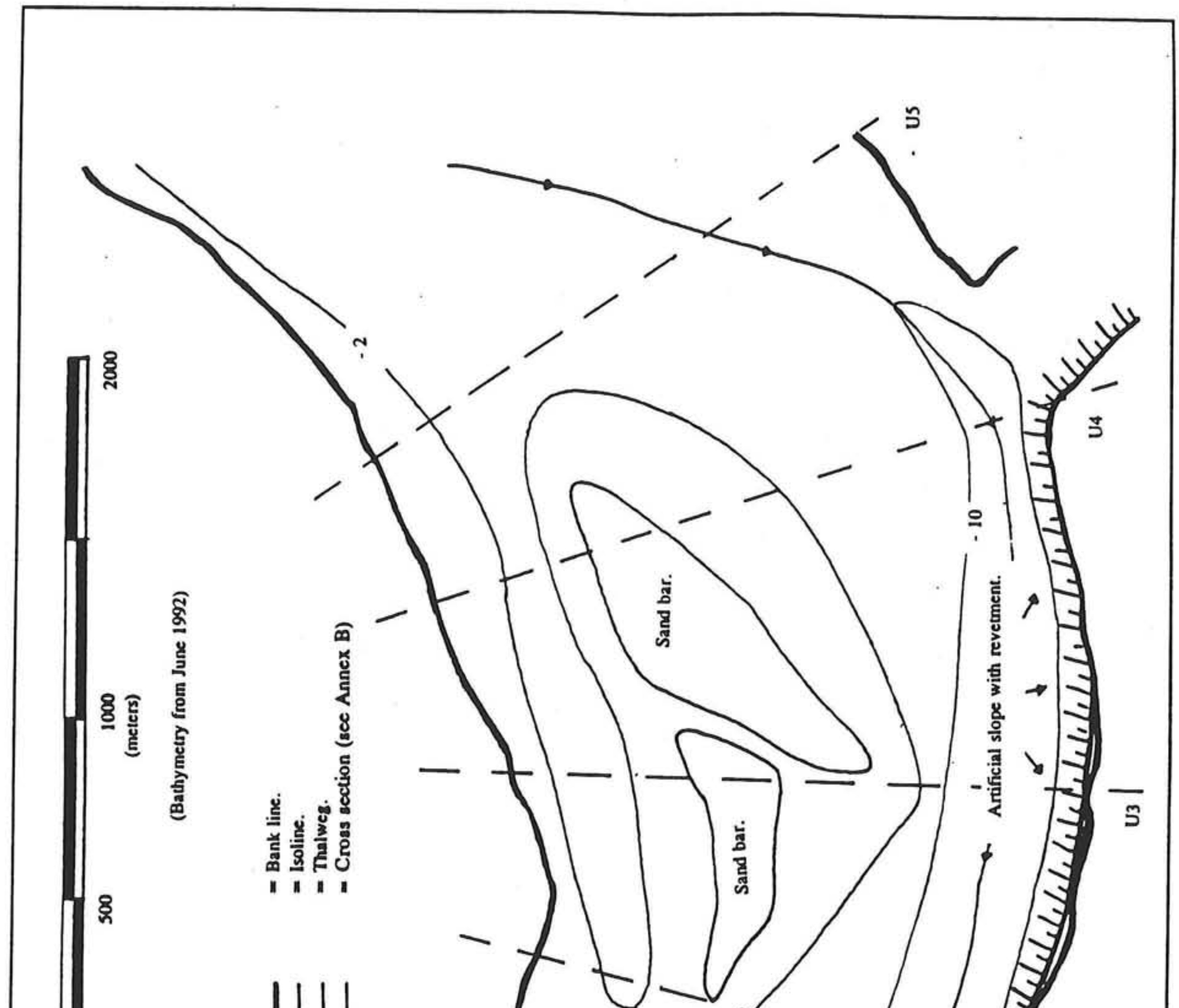

Figure II.7: Layout alternative I.

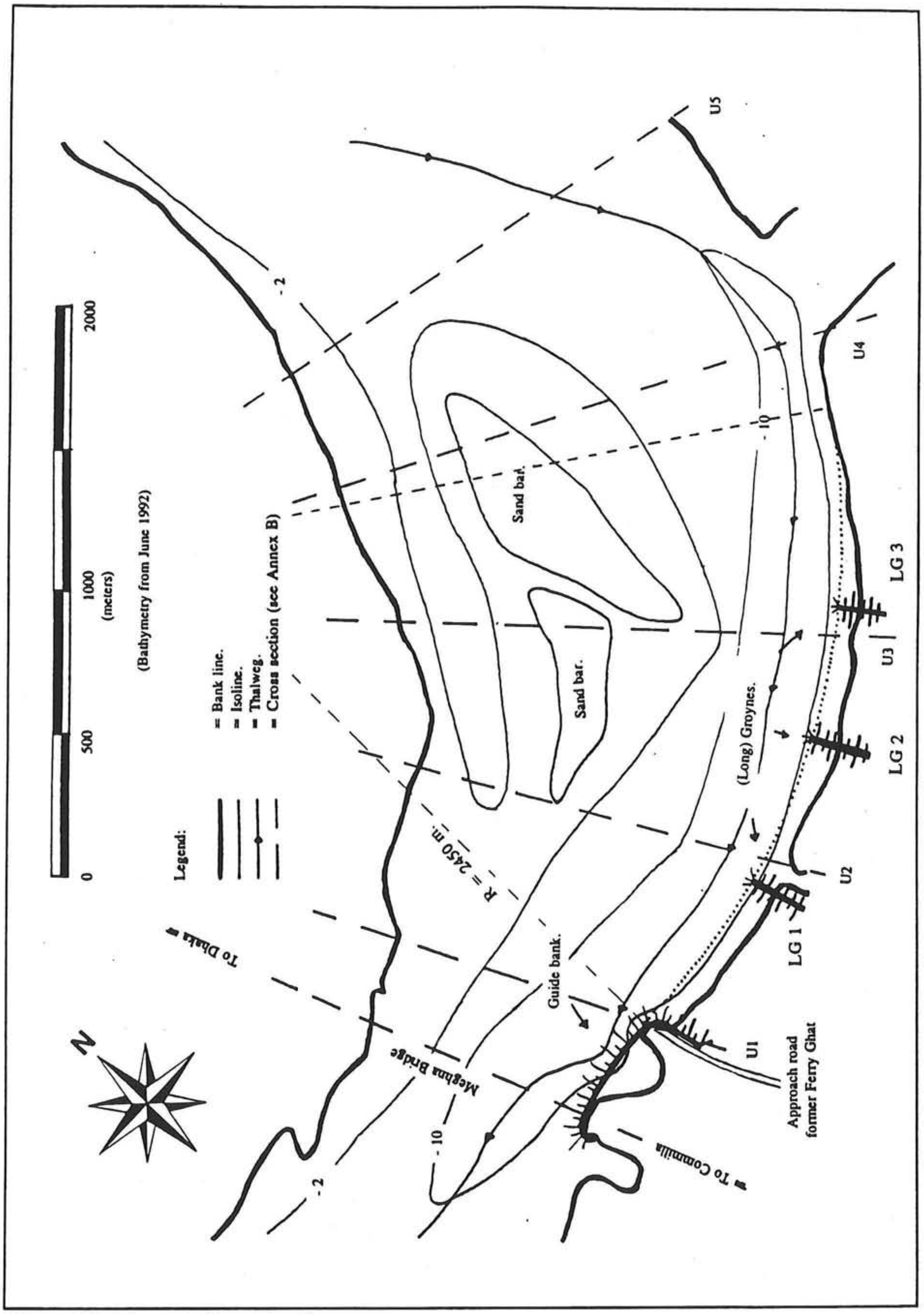

Figure II.8: Layout alternative II.

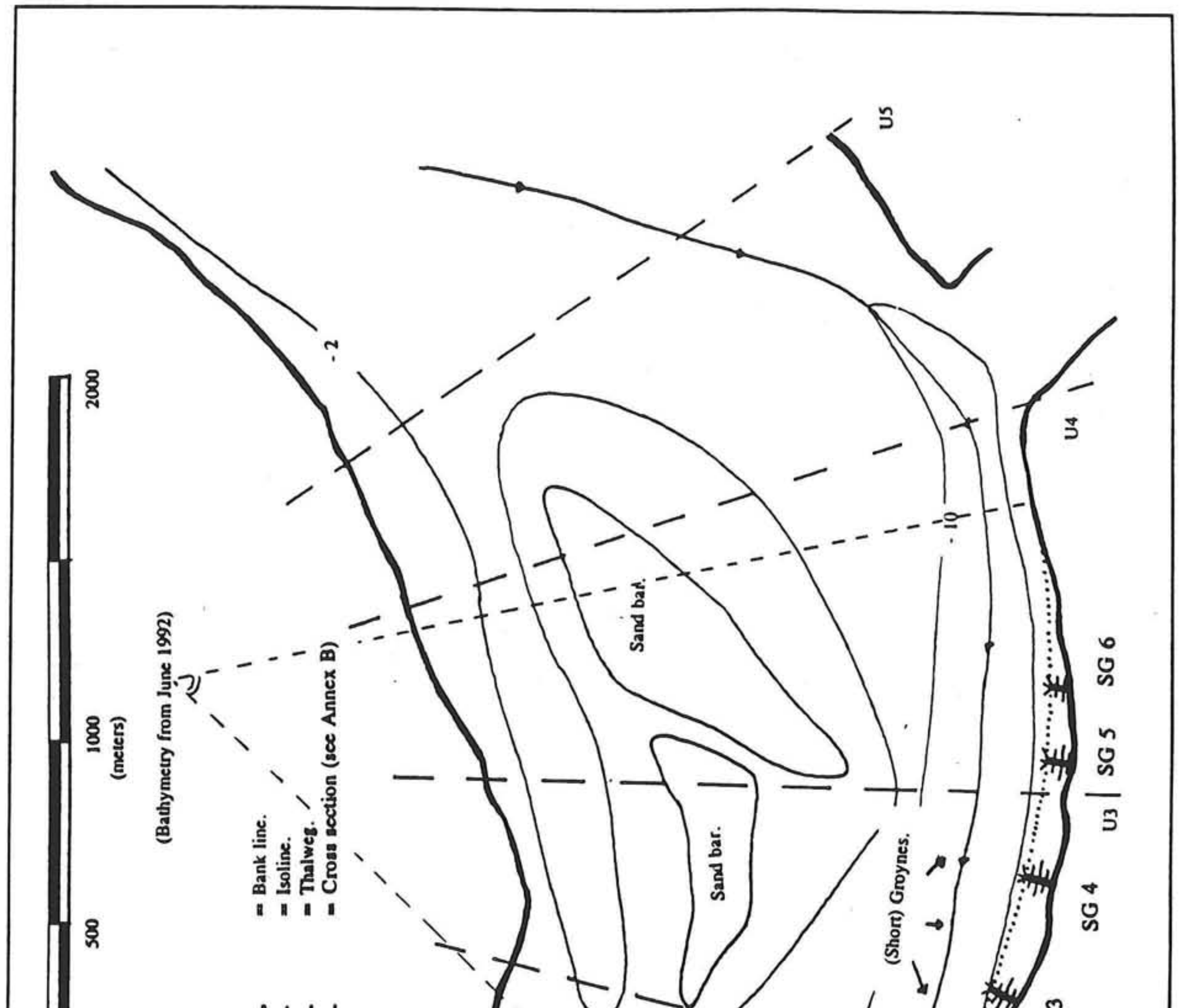

Figure II.9: Layout alternative III.

7 CONSTRUCTION MATERIALS

7.1 General

7.1.1 Introduction

In this Chapter the construction materials for the proposed structures foreseen in the most promising design layouts, as determined in the previous Chapter, will be determined.

In this Chapter the evaluation of the construction materials will be divided here into three parts. These are:

- construction materials for the core,
- construction materials for slope protection and
- construction materials for the toe section.

Before discussing the possible construction materials first these structure parts are defined and their functional requirements are discussed.

- The **core** is the body of the structure. Its main task is to form the mass. If it is vulnerable to any load (which depends on the construction material it is made of) it is protected by the slope protection.
- The **slope protection** forms the skin of the structure. It must protect the inner body against all possible loads as current and wave attack. Slope protection is performed by a revetment. For the protection of structures it is possible to apply "closed" or "open" revetments. In view of the differences between ground water tables and low water stages (see also Annex C) a "closed" protection is not preferred. Therefore only "open" type revetments are considered here. When an "open" type revetment is used a filter can be required also. This depends on the material used for the core and slope protection.
- The **toe** is the basis of the structure. Here the connection between the riverbed and the structure is made. However (the level of) this interface can change due to scour in the river bed or settlements of the structure etc. Extra attention must be paid to its construction and therefore this part of the structure is treated separately for its possible construction materials.

7.1.2 Criteria

The criteria to which the various alternatives for the construction materials will be evaluated can be divided into two groups:

Technical/construction criteria:

1. **Strength** (short and long term resistance to the hydraulic loads; waves and currents).
2. **Durability** of the material, resistance to biological and/or chemical influences, long term behaviour.
3. **Flexibility** to settlements, the material must be able to conform itself to deformations in such a way that no gaps or other big damages occur.
4. Flexibility to **maintenance**, if in a later stage maintenance must be conducted, it is preferable that elements can be repaired or replaced easily within the entire structure.
5. **Technical complications** arising from overlaps (slope protection), application in curved areas,

compatibility of the construction materials mutually, etc.

6. **Quality** assurance and control.

Socio/economic criteria:

7. Safety against **vandalism and theft** of the construction material. This yields especially for the slope protection materials in the upper section of the structure, as these are the easiest to grab. The attractiveness of the materials of course also plays a role.
8. **Compatibility** of the proposed construction materials and used equipment to the already locally available techniques and skills.
9. **Costs/availability**, it is clear that these aspects are closely linked, if the material is not easily available the costs will rise, also the demand on foreign currency is an important aspect here.

These criteria are considered when evaluating the various alternative materials for the various structure parts (core, skin and toe). However, not in every evaluation they will be as dominant. When for instance evaluating the various core materials most of the criteria are not directly of interest. Therefore this evaluation is only done by discussion keeping the criteria in mind.
As at the other hand for the slope protection works the number of alternatives is quite high and also almost all of the criteria are applicable here use is made of evaluation matrices. When exploring the various material possibilities for slope protection (Section 7.3) these matrices are presented.

7.2 Core materials

7.2.1 Introduction

As in the alternatives several "detached" or "semi-detached" hydraulic structures, as the guide bank and the groynes, are foreseen it might be interesting to discuss also core materials other than sand. Here with "detached" is meant that the structure is not connected to the hinterland.
Before discussing the various alternatives for the core material it is practical to have any idea of which amount is needed. The options for core material only must be evaluated for the guide bank and for the alternatives 2 and 3. When the river bank is revetted this is done on the existing "core material" which is the river bank material (Layer II and layer I type, see Annex C). However, some kind of "back filling" might be required if the revetment is placed "advanced".

The following (preliminary) volume estimates are made:

As a first estimate the volume of the core of the projected guide bank is estimated to be 500,000 m^3 (assuming an outer slope of 1:3 and an inner slope of 1:2 and a crest width of 6 m.). This volume will be bigger when the whole "bay" (between the Bridge abutment and the former ferry ghat) is filled, the estimated volume then is around 800.000 m^3.

For alternative 1, revetting the river banks, a relatively small volume might be needed to fill out the slopes. A more exact estimate is not made here.

For alternative 2 and 3 a volume of core material of around 350,000 m^3 for the groynes must be foreseen.

7.2.2 Alternatives

To serve as core material several options are given below:

- **Sand**; locally two main sources are available for providing core material: the river bank and the river bed. From the river bank in fact only the various soil types as determined in Annex E are available. At the location of the Meghna Bridge only soil of the Layer I and the Layer II type are present. From the river bed also two sources are available; the normal (deep) river bed and the sand bar in the bend just upstream from the Meghna Bridge. Geotechnical aspects are discussed further in Annex E.
- **Rock**, with Bangladesh mainly being a deltaic area, which is formed by deposition of alluvial sand, rock is quite scarce. As a consequence the prices for rock are quite high and therefore it is not advisable to use rock in large quantities.
- **Boulders** are more widely available in Bangladesh. However, as mentioned already, when ordered in large amounts (over 100,000 m^3) it can also become a problem. In view of the large quantities needed for a core also this option is not advisable.
- A cheaper solution is to produce a remplacant for rock or boulders being **sand cement stone blocks**. Though this is not common use yet, it must be possible to produce this locally.
- Also **concrete blocks** can be applied but these will be more expensive of course.

7.2.3 Discussion

As Bangladesh is a country where the ground mainly consists of alluvial deposits boulders and rock are quite scarce here. Therefore it is not advisable to use these as a core material because this would lead to unnecessarily expensive structures. Also the use of cement blocks is not advisable as this leads also to an expensive structure which is not necessary. For both these options the actual strength of the core would be considerably exceeding the required strength.
Therefore only sand or sand cement stone blocks remain as serious options.

When a sand core is considered however, for the projected guide bank in the under water section (below 1.00 m+PWD, see Section 8.1.3) sand can only be used as hydraulic fill. Within the Short Term Study also a trimmed and revetted sand slope with a steepness of 1:3.5 was proposed. This however is not possible for this particular project for the following reason:

- For the under water trimming of sand slopes "high tech" dredging techniques are required which are not available within Bangladesh. As therefore a foreign contractor is needed construction becomes very costly. When taking in account the dredged volume it must be concluded that for this (small scale) project this option can be regarded as non-feasible (Note: in the Short Term Study this slope trimming was proposed for much larger structures and also tendered as a package deal for more than one location and therefore the dredging volumes and the surfaces to trim were much larger). However, normal dredging techniques are available within Bangladesh.

If however the outer slope is formed by another construction material (for instance containment bunds, consisting of coarser material) sand can be applied as hydraulic fill.

Sand cement stone blocks can be regarded as an artificial remplacant for quarry run. It is mentioned already that in the alluvial plane which is Bangladesh rock and boulders are quite scarce. Therefore sand cement stone blocks might be an interesting alternative construction material.
Though sand cement stone is not yet common use as construction material for hydraulic structures

it may be an interesting option here as for the core material no long term resistance against wave or current attack is required. Sand cement stone blocks with a D_n of about 0.25 m. ($W \approx 30$ kg.) can be produced from sand of a normal grading and with a D_{50} of about 0.2 mm and a cement percentage of about 4%. The D_{50} of the river sediment at the location of Meghna Bridge is 0.14 mm while the soil layers in the bank are even finer. From the data presented in the S.T.S. however, it is revealed that the sediment more upstream in the Upper Meghna reaches a D_{50} of 0.2 mm. Therefore the sand, required for the production of sand cement stone, can be dredged more upstream. As most sand types, available in Bangladesh, have a small grading a higher cement percentage might be required.

Production of sand cement stone blocks can be done everywhere where there is enough space available.

The durability of sand cement stone (esp. when abrasion is consdidered) is not very high but in fact also not important for this particular application. Therefore it is decided to elaborate also structures with cores consisting of sand cement stone. It must be emphasized however that, due to these aspects, sand cement stone blocks can not be used as slope protection material (see also Section 7.3 where slope protection options are discussed).

7.2.4 Conclusion

The most interesting options for core materials are sand applied as hydraulic fill and sand cement stone.

From a technical point of view there is a distinction. As sand is very vulnerable to "leaching" it must be covered by an adequate filter. Sand cement stone blocks however, only require the construction of a good top layer which can resist the hydraulic loads.

Both these options will be kept in consideration. As the choice between these two options here is not taken yet also the determination of the most preferable top layer (slope protection), both for the under water- and the upper slope, will be evaluated for both possibilities of core material.

For all proposed structures both the application of sand cement stone or sand as core material will be elaborated further. However, one exception is made;

For the advanced river bank protection only sand is considered as core material. As parts of this structure are constructed on the present bank line application of sand cement stone for the places were the bank line is to be "trimmed" would introduce a lot of transitions in the structure which are always very vulnerable. Also the advantage of the application of sand cement stone as core material, with its steeper possible slope angle, is of no use in this case as the slope is governed by the (non advanced) parts where sand is the core material.

7.3 Slope protection

7.3.1 Introduction

As in the previous Section for the core two material types were selected to investigate further here also a distinction must be made.

When applying a hydraulic fill measures have to be taken to prevent the sand from being soaked out.

This can be done by means of a filter. The requirements for such a filter are discussed later in Section 7.3.4.

On the other hand, if a body of sand cement stone is taken only a slope protection is needed which protects the body of the structure against the hydraulic loads. Here however, during construction, still a filter can be required at the toe of the structure to prevent heavy erosion before the toe section is completed.

In this Section, where the slope protection is discussed, both these options are treated separately. First the slope protection when a hydraulic fill is applied is discussed and next the options for slope protection for a core of sand cement stone.

7.3.2 Slope protection alternatives when a hydraulic fill is applied

A distinction is made between the under water slope, for which construction always has to take place under water and the upper slope where construction in the dry season can be done in dry circumstances.

7.3.2.1 Under water slope

When a hydraulic fill is applied in the under water zone the slope of the structure can be determined making use of a helping structure. The various functions (macro slope stability, protection against hydraulic loads, prevention of outflushing of the core material) can be split or combined within parts of this helping structure. Two options are relevant here:

- **Containment bunds** can be applied to provide a stable outer slope. Bunds are constructed by dumping coarse material which is (temporarily) resistant against the (small) hydraulic loads which occur during the construction (dry) season. For large depths dumping is done making use of a dumping pipe to be able to position the destination more exact. After construction of the first containment bund hydraulic backfilling is done. Next the following bund is constructed etc.. The height of the bunds depends on the applied construction method and usually varies from 1 to 3 m. A small height will save a lot of (expensive) material. The material for the containment bunds can be stone debris as crushed rock or sand cement stone etc.. In theory containment bunds can be constructed with a slope of about 1:2. But in the final situation a protective layer must be provided at the outer side to protect the structure against hydraulic loads and outflushing of the hydraulic fill (filtering). This can be done by fascine mattresses with dumped rock or boulders and to ensure their stability a maximum slope of about 1:3 can be applied. Though boulders are a little bit less stable than rock they are preferable as protective layer because they are locally available and therefore less expensive.

- Another option is the placing of **gabions**. Gabions provide the stable outer slope (up to 1:2) and are also resistant to the hydraulic loads which can occur in the final situation. So here two functions are combined here within the helping structure. To prevent the hydraulic fill from outflushing in between the gabions and the hydraulic fill a filter layer must be foreseen. Here a geotextile is the most practical and economic.

Comparing these options with the criteria, set in Section 7.1.2, results in the following Table:

Table II.3: Comparison of alternatives for under water slope protection for sand bodies.

Alternative / Criterion	1	2	3	4	5	6	8	9
Containment bunds, covered with boulders or rock and a geotextile in between	+	o	+	o	o	o	o	A
Gabions, placed on a geotextile	+	o	+	o	-	-	+	H

The numbers of the criteria correspond to the numbering as used in Section 7.1.2. From these criteria number 7 (safety against vandalism and theft) is not applicable to under water sections and therefore not used here.

The marks in the comparison Table signify the following:

+	no problems or problems of a limited (acceptable) nature expected,
o	certain problems expected which are solvable but require special attention,
-	serious problems expected,
A	average (cost),
H	high (cost).

Below an explanation is given for the criteria for which the alternatives differ:

- The **technical complications** which are foreseen for the gabions is the placing of the geotextiles below these gabions and the application of gabions in curved areas.
- The difference in **quality assurance** here is due to the fact that gabions have to be placed on the geotextiles which were placed before. The transition between these two items must be good. This induces the risk of the occurrence of gaps in the structure. These gaps are not easy to detect under water.
- Concerning the **compatibility** to the locally available construction techniques and equipment it can be noted that placing of gabions is a known technique within Bangladesh which probably can be done with local equipment, the only problem might be the depths. For the construction of containment bunds however, special equipment is needed, especially for the positioning of the dumping pipe.
- The difference in **costs** between the two options is due to the fact that gabions are filled with material which is resistant to the hydraulic loads while in the other option such material is only foreseen as a cover layer. This results in a "wastage" of material when applying gabions. Also when bricks are used as filling material for the gabions, costs will still be quite high.

Conclusion:

From the Table above it can be seen that containment bunds covered by a fascine mattress with a protective layer is the most attractive solution when a hydraulic fill is applied.

7.3.2.2 Upper slope

For the upper slope, where construction can be done in the dry, more options are possible. With sand a slope of 1:3.5 can easily be shaped and revetted. Besides the options which are listed as possibilities for under water slope protections the following options for slope protection are also possible:

- Dumped **boulders, rock or concrete blocks**, placed on a geotextile or a granular filter.
- **Block mats** placed on a geotextile. Connections between the blocks and geotextile can be made in various ways.
- **Fabri-form concrete mats**, these are cushion-like geotextile mats, filled with concrete.
- A **pitched revetment** placed on a granular filter or a geotextile with outfilling material.
- Also **bound or grouted aggregates** like open stone asphalt can be used.

A presentation of the comparison of the alternatives in matrix form results in the following:

***Table II.4**: Comparison of alternatives for upper slope protection of a sand body.*

Criterion / Alternative	1	2	3	4	5	6	7	8	9
Gabions on a geotextile[A]	+	o	+	o	o	+	o	+	A
Boulders on a geotextile	o	+	+	+	+	+	-	+	A
Rock on a geotextile	o	+	+	+	+	+	-	+	A+
CC-Blocks on granular filter	+	+	+	+	+	o	o[B]	+	A
CC-Block mats	+	+	+	-	o	+	o	-	A+
Cellular concrete/geotextile mattresses	+	-	-	-	o	+	+	-	A+
Open stone asphalt on a geotextile	+	+	+	+	+	+	+	o	A

Note A) For this option within the comparison it is assumed that this is also applied in the lower section.

Note B) The safety of dumped concrete blocks against vandalism and theft can easily be modified with the applied block sizes.

The numbers of the criteria correspond to the numbering as used in Section 7.1.2.

The marks in the comparison Table signify the following:

+	no problems or problems of a limited (acceptable) nature expected,
o	certain problems expected which are solvable but require special attention,
-	serious problems expected,
A	average (cost),
A+	above average (cost),
H	high (cost).

Concerning the fulfilment to the various criteria the following remarks can be made:

- In the upper section the **strength** is determined by the resistance to wave loads as these are the prevailing hydraulic loads here. The resistance of boulders and rock to wave loads is not optimum. Open stone asphalt and CC-blocks have a good resistance against waves.
- Differences in **durability** are mainly due to the use of iron (gabions) or the exposure of geotextiles to biological or chemical influences (cellular concrete/geotextile mattresses).
- When comparing the **flexibility** to settlements special attention must be paid to the cellular concrete/geotextile mattresses. From the present situation at the Meghna Bridge it is already known that this is one of the most vulnerable aspects of these mattresses. Also a pitched revetment might induce some problems concerning this matter.

- The differences in flexibility to **maintenance** are quite trivial. Cover layers from dumped materials (boulders, rock) or materials which also can be added or replaced quite easily as open stone asphalt are very easy to maintain, compared to those materials which are much more rigidly inter connected.
- These more rigidly inter connected materials will also induce some **technical complications**, a good example is their applicability in curved areas.
- **Quality assurance and control** can give some problems when a granular filter is used.
- Concerning the safety against **vandalism and theft** it is remarked already that for concrete blocks this easily can be anticipated by modification of the block sizes. For boulders and rock however this is less easy and therefore these are the most vulnerable to vandalism or theft. The other applications of CC-blocks (pitched and mats) are assumed to be less vulnerable as they are more or less fixed in position.
- Concerning the **compatibility** of the needed construction techniques and equipment it must be remarked that concrete block mats and the cellular mats can not be produced with any local source available. The application of open stone asphalt however requires only a slight modification of the available bitumen plants and the construction equipment. This could even have an extra additional value for Bangladesh in general. There is also the possibility that meanwhile this is done already within the framework of the Jamuna Bridge Project.
- Concerning the **costs** the explanation of the differences is quite simple: high tech materials require foreign currency and are very expensive and as in Bangladesh concrete and boulders are more easily available than rock these are also less expensive.

Conclusion:

From the comparison above it can be seen that open stone asphalt on a geotextile is the most attractive solution for the protection of the upper slope when sand is applied as core material.

7.3.3 Slope protection alternatives for a body of sand cement stone blocks

Though sand cement stone is a coherent material it is not resistant against hydraulic loads on the long term. Especially the scouring effect of a sand-loaded flow can cause reasonable damage. Therefore the core must be covered with a separate protective layer. As no filter is required only protection against the hydraulic loads must be provided.

As for the upper slope section more alternatives are available (construction can be done in the dry) here also a distinction is made.

7.3.3.1 Under water slope

The alternatives for the under water slope protection are:

- **Concrete blocks**, these can be placed straight upon the sand cement stone blocks of the core.
- **Rock or boulders** without any filter layer beneath might also be sufficient.
- Also the use of **gabions** can be considered.

A comparison results in the following:

Table II.5: Comparison of alternatives for under water slope protection for a body of sand cement stone blocks.

Alternative / Criterion	1	2	3	4	5	6	8	9
CC-blocks (dumped)	+	+	+	+	+	o	+	A
Rock (dumped)	+	+	+	+	+	o	+	A+
Boulders (dumped)	+	+	+	+	+	o	+	A+
Gabions	+	o	+	o	-	o	+	H

The numbers of the criteria correspond to the numbering as used in Section 7.1.2. From these criteria number 7 (safety against vandalism and theft) is not applicable to the under water section and therefore not used here.

The marks in the comparison Table signify the following:

+	no problems or problems of a limited (acceptable) nature expected,
o	certain problems expected which are solvable but require special attention,
-	serious problems expected,
A	average (cost)
A+	above average (cost)
H	high (cost)

The differences between the various options here occur for the following criteria:

- Because of the application of iron the **durability** of gabions must be expected to be limited.
- The flexibility to **maintenance** of gabions is a little bit worse than for the other alternatives.
- **Technical complications** arise when gabions are applied in curved areas.
- It is mentioned several times already that rock and boulders are quite scarce in Bangladesh. Therefore the **costs** of CC-blocks is expected to be the lowest.

Conclusion:

From the discussion above it can be seen that CC-blocks are more attractive than the other options for the under water slope protection for a core body of sand cement stone.

7.3.3.2 Upper slope

Besides the alternatives which were already mentioned for the under water slope the following alternatives are considered:

- **Block mats** could be applied. However, as the surface of the core will not be flat itself some fill in material must be placed in between the core and the skin.
- Probably it might also be possible to increase the resistance against hydraulic loads of the sand cement stone blocks by grouting thus creating a skin layer of **grouted sand cement stone.**
- But also "normal" **bounded or grouted aggregates** as open stone asphalt can be considered.

A comparison is presented in the following Table:

Table II.6: Comparison of alternatives for upper slope protection for a body of sand cement stone blocks.

Alternative / Criterion	1	2	3	4	5	6	7	8	9
CC-blocks (dumped)[A]	+	+	+	+	+	+	o[B]	+	A+
Rock[A]	o	+	+	+	+	+	-	+	H
Boulders[A]	o	+	+	+	+	+	-	+	A+
Gabions[A]	+	o	+	+	o	+	o	+	A+
CC-Block mats	o	o	+	-	-	+	+	-	H
Grouted sand cement stone blocks	o	o	-	+	+	o	+	+	A
Open stone asphalt	o	o	+	+	-	+	+	o	A+

Note A) Also here for these alternatives it is assumed that these are applied in the lower section as well.

Note B) As mentioned already the safety of concrete blocks against vandalism and theft can easily be modified with the applied block sizes.

The numbers of the criteria correspond to the numbering as used in Section 7.1.2.

The marks in the comparison Table signify the following:

- \+ no problems or problems of a limited (acceptable) nature expected,
- o certain problems expected which are solvable but require special attention,
- \- serious problems expected,
- A average (cost)
- A+ above average (cost)
- H high (cost)

Concerning the fulfilment to the various criteria the following remarks can be made:

- For boulders and rock here the same holds as when applied on a hydraulic fill and a geotextile. But also the **strength** of covering layers as block mats, grouted sand cement stone blocks and open stone asphalt here is expected to be lower than usual because no good sub basis is available for these materials. As these covering layers are much less permeable than the sub basis pressure differences can arise which result in higher loads and therefore the relative strength decreases.
- Also the **durability** of the covering layers as block mats, grouted sand cement stone and open stone asphalt will be lower than usual. The durability of grouted sand cement stone is quite doubtful because sand cement stone on the long term is not very resistant against the abrasing forces of sand loaden flows.
- Grouted sand cement stone takes a particular place when the **flexibility** to settlements is regarded. When settlements occur the "protection layer" will crack thereby loosening its strength.
- The flexibility to **maintenance** of block mats is not very good. As the stones are fixed in position it will be difficult to replace them.

- The **technical complications** for the application of block mats or open stone asphalt are due to the fact that both these options require a flat sub basis. With a core of sand cement stone however, this is not the fact. For the gabions complications are foreseen when they are applied in curved areas.
- **Quality control** for grouted sand cement stone is not easy to conduct.
- For the safety against **vandalism and theft** and the **compatibility** to the locally available construction equipment the considerations are the same as for the upper slope protection alternatives when a sand body is applied. Only grouted sand cement stone is new here. As the pieces are fixed in position safety against vandalism and theft is assured and the required construction equipment is very simple.
- Also the **costs** are very similar to as when applied on a sand body. The added alternative of grouted sand cement stone is not very expensive.

Conclusion:

For the protection of the upper slope of a core body of sand cement stone blocks the use of concrete blocks is the most attractive alternative. However, as concrete blocks are also a very attractive construction material for other purposes than slope protection the sizes of the stone blocks must be taken in such a way that vandalism or theft is prevented.

7.3.4 Filters

Filters can be required in various cases. The most common is if there is a danger for outflushing of material out of the structure. This is the case in this project where a sand body is applied. Therefore in the next Sections the requirements for these filters are discussed. However, also during construction a filter can be required to prevent heavy scour. As the determination of these is more a practical case this will be done during the detailing of the designs (Chapter 8).

7.3.4.1 General

When the core material is vulnerable to outflushing (sand body) as part of the slope protection a filter must be constructed. The demands for such a filter are the following:

- Prevention of the sub-soil from outflushing. A filter assures the micro stability of a structure.
- Prevention of development of pressure within the bank/dike.
- A filter must redistribute (spread) the loads.

Based on these functions some general requirements can be listed which the filter material (and the top layer/revetment) should meet.

- It must be impossible that the filter material flushes out through the pores of the top layer.
- A filter and its sub basis must be internally stable. This means that the finer fractions of a layer can not move through the wider pores in the filter. As the grain distribution is wider (good grading, bad sorting) the danger for internal instability increases.
- As the complete structure needs to be flexible to deformations this is also required for the filter. "Arching" is not permitted, as this will lead to soil transport through the created holes. This could cover the existence of "arches" but can lead to a sudden collapse of the structure over a big surface.
- To prevent "pressure building" within the dike the filter must be around five times more

permeable than the base (rule of thumb). A filter has to be designed following the filter rules to assure that its permeability increases from bottom to "skin" but that also filling of the filter by sand from the base is prevented.

- It can be required that the filter is able to resist the hydraulic loads to which it can be exposed during construction.

7.3.4.2 Filter types

Several filter types are possible, namely: a granular filter, a geotextile or a thick layer of broadly graded natural or waste products. They are discussed below:

- a **granular filter** consists of several layers of graded material, increasing in diameter from the sub-base to the "skin". Such a filter is relatively expensive. Especially for the under water part it is difficult to construct such a filter within the permissible tolerances,
- a **geotextile** is a membrane which performs the filter function completely. It is sand-tight but permeable for water. A distinction can be made between so called "wovens" and "non wovens" which refers to the way these geotextiles are produced. Both these types have their specific merits. A geotextile can be combined with a layer of graded stone which has the function to damp the internal hydraulic loads.
- **A thick layer of broadly graded natural or waste products** as minestone, slags, silex etc. can also be applied. The thickness of such a layer can range to 1.0 m for high hydraulic loads. The compaction and composition have to be controlled according to criteria on internal stability.

7.3.4.3 Applied filter type

As the largest part of the protected slopes will be the under water part usage of a granular filter is not preferable. Though the construction of a thick layer of broadly graded material will be a bit easier to conduct under water also this alternative is not preferable. Another disadvantage for this option is that such an alternative requires a lot of material which is rather scarce in Bangladesh and therefore it will be quite costly. Therefore whenever the construction of a filter will be required within this project use will be made of geotextiles.

It is assumed that for this particular project importing geotextiles, which is normally forbidden in Bangladesh, will be allowed.

7.4 Toe of the structure

Before discussing the material alternatives first a discussion is held on the type of toe protection which is going to be applied for the various structures.

7.4.1 Type of toe protection

When the toe of a hydraulic structure is not foreseen at the expected maximum scour level it must be protected against further scour. Four possibilities for such toe protection are available. These are:

i. a launched or **falling apron** (a flexible stone protection)
ii. providing a **mattress** in front of the structure to cover newly scoured slopes.
iii. providing an impermeable curtain of **sheet piles**.
iv. checking and elimination of the scouring force for instance by constructing **submerged sills** in front of the bank which help to stop the deepening of the bed.

As it is known that the bed forms of all rivers in Bangladesh, including the Upper Meghna River, are absolutely not stable account will have to be taken for changes herein. Therefore the toe of structures must be flexible to be able to cope with these phenomena.
From the alternatives mentioned above it is clear that a solution with sheet piles is absolutely not advisable, this has also become painfully clear with the presently existing hydraulic structures (the left abutment) at the location of the Meghna Bridge.
For the fourth alternative an extensive (modelling) study has to be elaborated to check its effectiveness and working.

Therefore only the first two alternatives remain as serious options. The main differences between these options are the following:

- a block mattress is sand tight while a flexible stone protection (falling apron) is not sandtight.
- a block mattress can resist forces in horizontal direction. As the material in the falling apron is not mutually connected here no forces in horizontal can be resisted.

The working of both protection types is shown in Figure II.10. The movement of sand is marked with arrows.
The processes depicted in this Figure can be described as follows:

- **For a block mattress** erosion starts at the tip of the mattress in the unprotected sand, thereby creating a scour hole in longitudinal direction, starting from the fully protected part. This results in steepening of the slope under the protection untill the critical slope angle is exceeded. From this moment the grains below the edge of the mattress start moving in lateral direction to the scour hole. The block mattress will lower at an inclination equal or even steeper than the critical slope angle for the sand (model tests, conducted in the framework of the Jamuna Bridge Studies, revealed a maximum slope angle of 1.5 : 1 (V:H), see ref. [12]). This mechanism goes on untill the erosion process in the scour hole stops and the lateral slope angle (α) is stable. This will result in a very steep angle α in the final state which results in a dangerous situation for loosely packed sands in combination with severe loading conditions like earthquake shaking.
- With **the flexible stone protection (falling apron)** also a scour hole will develop in the unprotected part in front of the tip. However, as the falling apron itself is not sand tight, also erosion will take place in vertical direction through the stones leading to a gradual lowering

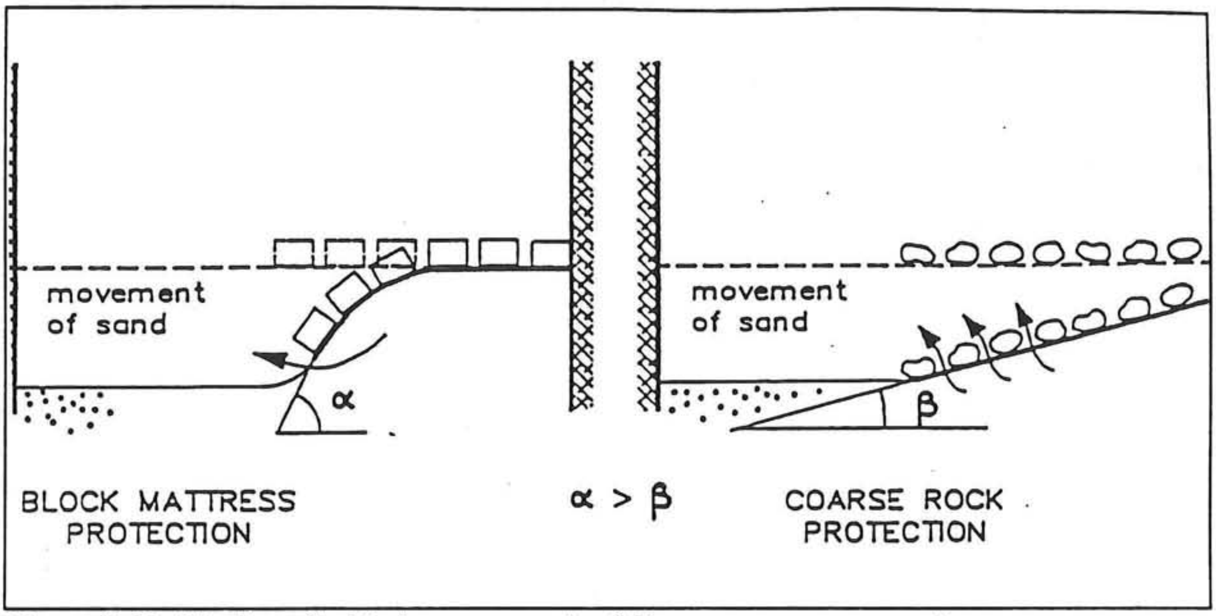

Figure II.10: Working of a block mattress and a falling apron as toe protection.

of the protected area (see the Figure above). As the scouring in the unprotected part goes faster and the erosion intensity increases as the waterdepth increases, a slope inclination arises in cross-sectional direction towards the scour hole. This slope inclination will induce sliding of (a part of) the slope protection thereby stopping (slowing down) the lateral shifting of the scour hole to the structure's toe. Angles steeper than the angle of repose of the underlying material will not occur (the model tests, conducted in the framework of the Jamuna Bridge Studies, revealed a maximum slope angle of 1 : 2.5 (V:H), see ref. [12]). Therefore the risk for geotechnical failures here is much lower.

From this evaluation it is concluded that a falling apron is the best type of toe protection for the structures foreseen in this project. The falling apron is discussed and detailed further in the next Chapter (Section 8.4).

7.4.2 Material alternatives

The most secure method to provide future coverage of the slope of the scour hole is to provide loose granular material. 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. The grading also must contain a sufficient quantity of material suitable to individually resist the current forces.

It is not preferable to provide a geotextile as a filter in a falling apron. This will badly influence the launching. Another danger arises if the material under the geotextile is eroded. The geotextile will be loaded very heavily by the apron material above and will crack.

There are not too many alternative materials which can be applied in the falling apron section. The options are:

- **Graded rock,** this alternative is very well resistant against all occurring forces but also quite expensive.
- **Graded boulders,** this alternative is almost as resistant as the rock alternative but less

expensive (as boulders are available in Bangladesh). A problem might arise if large diameters are required.

- **A combination of concrete blocks with graded sand cement stone blocks.** The concrete blocks must perform the flow resistance function while the graded sand cement stone blocks provide the under layer (filter). Sand cement stone alone is not expected to be very resistant against current attack on the long term. This might even be worse for sand loaden flows which are linked to scouring phenomena. Therefore graded sand cement stone without CC blocks is not a satisfying alternative.
  It must be remarked here that it will not be easy to obtain the proper grading here and to dump the apron material the right way. Due to the differences in density of concrete and sand cement stone also the launching behaviour of this mixed material is doubtful.
- From the previous option it can also be considered to improve the quality of sand cement stone blocks. This results in using **low quality CC-blocks** (with for instance broken bricks as aggregate). The strength (durability) of these blocks is significantly higher than normal sand cement stone blocks (which could be used as core material). An advantage of this option is that the density of the material is constant and therefore the reaction to loads of the apron material is more uniform.

7.4.3 Conclusion

The optimal apron material is also dependent on the applied slope protection for the under water slope or the core material. To continue this materials into the falling apron section anticipates problems at the transition of these sections, which otherwise could arise, and limits the variety of used construction materials. Therefore two options remain. Their application depends on the applied material for slope protection and for the core of the structures. The options are:

- A falling apron section of graded boulders, if larger sizes are required also rock can be considered. The latter is more expensive however.
- A falling apron section of graded CC-blocks with brick aggregates. Strictly speaking this option introduces another construction material but the required techniques to obtain this material are already present when CC-blocks and sand cement stone are applied.

For the various structures the best combinations of construction materials are determined in the next three Sections. The dimensioning of the falling apron will be discussed more extensively in the next Chapter, where the designs are detailed, in Section 8.4.

7.5 Most promising alternatives for the various structures proposed in the alternatives

7.5.1 Guide bank

For the guide bank there is an additional consideration which influences the choice of the possible construction materials. It is preferable that the front slope (i.e. at the river side) of the guide bank structure is as steep as possible. There are two major reasons for this.

- At present a deep scour hole is formed in front of the former ferry ghat. With the steepest front slope the reach of the structure into this scour hole and the construction depth can be kept limited. This will decrease the final construction volume and thereby the costs.
- The other merit of a steep front slope is that the extra constriction of the river cross section

at the Bridge due to the structure is minimal. This will result in a minimum scour induced by the structure and will therefore also minimize the required volume of material in the falling apron section.

From the previous Sections it can be learned that there are two serious options for the guide bank structure:

1). A guide bank with a sand core. The entire "bay" is filled. At the river side the under water section is constructed with containment bunds covered by a fascine mattress with boulders. For the upper section, which can be built in the dry, a protection of open stone asphalt is the best alternative. As boulders are used for the protection of the under water section this is also the most attractive material for the falling apron section.

2). A guide bank with a core of sand cement stone. The bay is not filled further. For both the under water and the upper section the protective layer consists of loosely dumped concrete blocks.
The falling apron section consists of concrete blocks with brick aggregates. Because of the turbulence induced during construction a filter might be needed at the toe to prevent scour before the falling apron section is constructed.

Though in the second alternative the steepest front slope can be established it is possible that the first alternative is more economical. Therefore both these options are dimensioned into more detail in the next Chapter after which an evaluation is conducted.

7.5.2 Submersible groynes

In principle for the submersible groynes there are also two possibilities. These are almost similar to the alternatives for the guide bank. Though they are submergible the groynes still have an under water and an upper slope as well. The crest level of the groynes will be above 0.73 m+PWD (Low Water Level). This means that, during the low water season, the crest will be above water level.

The alternatives are:

1). Groynes with a sand core. The under water section is constructed with containment bunds covered by a fascine mattress with boulders. For the upper section, which can be built in the dry, a protection of open stone asphalt will be the best alternative. As boulders (on a geotextile) are used for the protection of the under water section boulders are also the most attractive material for the falling apron section.

2). Groynes with a core of sand cement stone. Here also for both the under water and the upper section the protective layer consists of loosely dumped concrete blocks. The falling apron section can be constructed with CC-blocks with brick aggregates. Here also between the groynes and the original river bed a filter is needed to prevent heavy scour during construction.

7.5.3 River bank revetment

As mentioned already in Section 7.2.4 for the alternative of river bank revetting only sand is regarded as an alternative for the "core body". As this is a typical attached structure the use of a core of sand cement stone for this particular alternative is quite ridiculous so here only one alternative is left.

- The river bank can be revetted by placing containment bunds. Behind the bunds an outfill is placed by an hydraulic fill. The bunds are covered by a fascine mattress with boulders and the falling apron section can also be constructed using graded boulders.

7.6 Final conclusion, overall alternatives

As it can be seen from Section 7.5.1 - 7.5.3 for the final stage of this project two alternatives for abutment protection and four alternatives for bend fixation remain (advanced bank protection, short groynes with a sand core, long groynes with a sand core and short groynes with a sand cement stone core).

There are two main considerations here to combine "total packages" for the final solution. These are:

i. To minimize the costs of the various alternatives it is preferable that the required construction techniques remain limited within one alternative. Therefore it is not advisable to combine a sand core body for the guide bank with bend protection works with sand cement stone cores or the other way around. As mentioned already in Section 6.1.4, the protection works must be constructed in one dry season and not in phases.

ii. If it is decided to construct groynes with a sand core, it will be much more economic to construct a limited number of long groynes than a large number of short groynes; the dredging volumes are almost similar but the required quantities of construction material (slope protection & apron section) reduce significantly when a limited number of longer groynes is applied. This construction material is much more expensive than some extra dredging (provided that dredging is done anyway).

From these considerations three alternatives remain to evaluate. These are:

- Alternative I:

A guide bank as abutment protection and bank revetment for river bend fixation in the bend upstream. Both are constructed with a sand core and slope protection consisting of boulders on a fascine matress in the under water section and open stone asphalt on a geotextile for the upper section. Graded boulders are used as apron material. For the general layout see Figure II.7.

- Alternative II:

A guide bank as abutment protection and a series of "two and a halve" (long) groynes for river bend fixation in the bend upstream. Here also both structure types are constructed with a sand core. Also the slope protection is similar: boulders on a fascine mattress in the under water section and open stone asphalt on a geotextile for the upper section. Again graded boulders are used as apron material. For the general layout see Figure II.8.

- Alternative III:

A guide bank as abutment protection and a series of "five and a halve" (short) groynes for river bend fixation. Here all structures are constructed with a core of sand cement stone. For the slope protection, in both the under water and upper slope, concrete blocks are applied. As apron material CC-blocks with brick aggregates are used. For the general layout see Figure II.9.

As a coincidence the number and characteristics of the layout alternatives, as determined in Section 6.6, are exactly the same.

These final alternatives are detailed in the next Chapter.

8 DETAILING OF THE ALTERNATIVES

8.1 Characteristic levels and zones

In this Section for the various alternatives all characteristic levels and some general dimensions are discussed and determined. With these levels as starting point the alternatives can be detailed further.

8.1.1 Design scour levels

The way the design scour levels are determined is extensively discussed in Annex F. Here only the final results are repeated:

Along the projected guide bank the design scour levels are taken constant at a level of 30 m-PWD. As the reach for bend fixation is quite long it is interesting from economical point of view to differentiate the design scour depth along the reach in order not to waste expensive construction materials. Therefore design scour levels for the revetment and groyne alternative are determined for each cross section and presented in the next Table.

Table II.7: Design scour levels for structures foreseen as river bend fixation works.

Cross section: Alternative	U4 (m PWD)	U3 (m PWD)	U2 (m PWD)	U1 (m PWD)
River bank revetment	- 30	- 25	- 20	- 25
Groynes	- 30	- 25	- 21	- 26

In between these cross sections the design scour levels will be linearly interpolated.

8.1.2 Top levels

It is remarked already that flood protection is beyond the scope of this project. Therefore the top levels for the various structures are related to the levels of the hinterland. This results in the following:

- For the **guide bank** the top level is taken equal to the Bridge abutment "plateau" which is 6.00 m+PWD.
- For the **river bank revetment** the top level is equal to the level of the present left river bank which is around 6.00 m+PWD at cross section U1 (the former ferry ghat) and around 4.00 m+PWD at cross sections U2 to U4.
- The crest levels of the projected **groynes** is also taken equal to the present bank level. In order to reduce hinterland directed flow a slope of 1:100 is applied for the crest of the part of the groyne which is located in the river bed. As no detailed topographical survey is available of the adjacent hinterland and to avoid the risk of inducing any outflanking process in the hinterland the crest level of the groynes is taken constant at the level of the bank line.

8.1.3 Slope zones

The level for the transition between the under water and upper slope can be governed by two demands. These are as follows:

i. If the upper slope protection consists of a material which can only be constructed "in the dry" (e.g. open stone asphalt) **dry construction circumstances** must be assured. Therefore, taking the dry season as construction season, its lower limit level must be above S.L.W.

ii. The transition level can also be governed by the **zones of wave and current attack**. As the simultaneous occurrence of both phenomena at their maxima has an extremely low probability the protection layer is dimensioned to the load which requires the largest block sizes.

- From Annex C it is known that the Standard Low Water (S.L.W) at the location of the Meghna Bridge is at a level of 0.73 m+PWD. Along the entire reach to be protected (the river bend upstream) the extreme head is around 0.11 m. Therefore at the upstream end S.L.W. will be approximately 0.84 m+PWD. To ensure the possibility of **construction in the dry** for the upper slope protection and to provide enough space for the connection of the material types the transition between under water and upper slope is taken at 1.50 m+PWD.

- As the highest top level is at 6.00 m+PWD and the water level with a return period of 100 years is 6.60 m+PWD the **zone of current attack** for the location of Meghna Bridge must be taken from the toe of the structures until the top (or crest).

- As the maximum water levels exceed the top of the structures for the **zone of wave attack** only the lower limit is of interest. A rule of thumb to determine this lower limit is Still Water Level (S.W.L.) - 1.5 to 2 H_s. As the thunder storms, which induce the design waves, mainly occur in April, the water level for april with a 10% non exceedance (which is 1.63 m+PWD, see Annex C) is taken being S.W.L. As the probability of occurrence of both phenomena (very low water level and storm with a return period of 100 years) at the same time is rather low the wave impact zone is taken at 1.5 H_s. With a wave height H_s = 0.96 m (see Annex C) this results in a lower limit for the zone of wave attack of 1.63 - (1.5*0.96) $\approx$ 0.00 m-PWD.

Therefore in the structural design of cover layers a distinction is made between three zones:

Zone I: From the toe of the structure until 0.00 m PWD.In this structure only current attack is present and construction always must be done "in the wet"

Zone II: Between 0.00 m PWD and 1.50 m+PWD. In this transition zone both current and wave attack are present, construction always must be done "in the wet".

Zone III: From 1.50 m+PWD until the top (or crest). In this zone both current and wave attack are present. Here construction can be done "in the dry".

The dimensions of the applied slope protection material will be depending on which phenomenon (wave or current) requires the largest sizes (see Section 8.3, where the required dimensions for the slope protections are discussed).

8.1.4 Toe levels

As the river cross sections vary considerably within the reach from the Bridge until cross section U4 (see also the design scour depths) it is impossible, and also not preferable, to determine one toe level for the entire reach.
From the discussion about the design scour levels it can be learned that it will be impossible to construct the structures until these scoured levels. Therefore the future scour holes will be dealt with by the falling apron section which is to be discussed later. The toe levels are taken at the present bottom level which is assumed to be the same as the bottom levels which follow from the bathymetrical survey from June 1992 (see Annex B). As these bottom levels at which the structure will be placed is relative to the slope applied here no values are derived yet. These will be determined when the alternative designs are finally determined (Section 8.5)

8.2 Slopes

In this Section the macro slope stability and geotechnical aspects for the various structures, as proposed in the alternatives, are discussed. A distinction is made between three failure modes, a straight failure plane, a circular failure plane and the bearing capacity of the subsoil.

8.2.1 Straight failure planes

Within the models available no account is taken for seismic influences. Therefore here the required safety factors, as discussed in Annex E, are not applied but a general requirement of $n > 1.2$ is set.

For protected slopes (which are considered here) with a ground water gradient the safety factor for the most critical situation is given by the following formula (ref [2]):

$$n = (1-i)\,\frac{\tan(\phi)}{\tan(\alpha)} \tag{2}$$

where:

n	= safety coefficient (-),
i	= gradient of the ground water flow perpendicular to the slope face, below the slope protection (-),
ϕ	= angle of internal friction of the core material (°),
α	= slope angle (°).

This formula must be applied to the situation with a sand body and a slope protection.

In an unprotected slope the gradient i may become 1 at the free water level thereby causing a slope collapse. For protected slopes however, this gradient reduces dramatically. In the S.T.S., where the ground water flow pattern was studied for the location of Bhairab Bazar, it was concluded that the expected gradient of the ground water flow for a protected slope (1:3.5) will be about 0.03 to 0.05. As the circumstances are similar to those at the location of Meghna Bridge the same value is applied here. Taking ϕ at 25° (the lowest value occurring) and α at 18° (slope 1:3) and i at 0.05 results in a safety value of 1.32. This is sufficient. A slope flatter or equal to 1:3 can be applied.

For the situation with a core of sand cement stone blocks the core is permeable and no ground water gradient is present (over the slope material). The safety factor is expressed by:

$$n = \frac{\tan(\phi)}{\tan(\alpha)} \tag{3}$$

Where parameters are similar as above.

This formula simply expresses that the applied slope angle should be flatter than the angle of internal friction. A safe estimate for the angle of internal friction is 40°. Applying a slope angle of 34° (1:1.5) results in a safety coefficient of 1.26. This is also sufficient. For this situation a slope flatter of equal to 1:1.5 can be applied.

When a geotextile is applied also a straight failure plane can be caused by the sliding of the geotextile along the slope. This is discussed in Section 8.3.5, where the requirements for the applied geotextiles are determined.

8.2.2 Circular failure planes

The required safety factors (n) are 1.5 for static loading and 1.1 for static and dynamic loading (including the seismic influences).
For the circular failure planes a distinction is made between three cases, two cases for a core of sand (explained below) and one for a core of sand cement stone blocks.

- Sand core

A distinction is made for a hydraulic fill (case 1) and sand in the same layer configuration as the present situation (case 2).

These cases and their computations are discussed more extensively in Annex E. Here only the results are repeated and presented in the Table below:

Table II.8: *Safety factors for various slopes with a sand core (circular failure plane)*

Criterion:	Slope 1 : 3 (V:H)		Slope 1 : 3.5 (V:H)	
Case:	n > 1.5	n > 1.1	n > 1.5	n > 1.1
Case 1	1.38	1.06	1.58	1.18
Case 2	1.45	1.11	1.67	1.24

From this Table it can be seen that a slope of 1 : 3.5 does match the design criteria for both cases.

- Sand cement stone core

In the previous Section it was already concluded that a slope of 1:1.5 is stable on the macro scale for straight failure planes.

Also a computation with Bishop's method is elaborated. The assumptions, as used in the simplified Bishop method, are not completely valid however. Within Bishop's method the shear stress is taken constantly present along the failure plane. This is the case for sand but for larger block sizes, as meant in this situation, the "shear stress" is much more related to interlocking phenomena and concentrated in specific points.

However, the computation is elaborated anyway, assuming an internal friction angle of 40° and no cohesion for the sand cement stone body.

The computation is presented and discussed further in Annex E and the results are as follows:

Table II.9: *Safety factors for various slopes with a sand cement stone core (circular failure plane).*

Criterion:	Slope 1 : 1.5 (V:H)		Slope 1 : 2 (V:H)	
Case:	n > 1.5	n > 1.1	n > 1.5	n > 1.1
Case 3	1.39	1.18	1.59	1.32

As the computer program will underestimate the safety coefficient (see Annex E) a slope of 1 : 1.5 is assumed still to match the design criteria.

8.2.3 Bearing capacity of the sub soil

To complete the story in Annex E also a check is made on the bearing capacity of the sub soil. This check revealed that the minimum safety factor for this aspect is 11. As this is much more than required no further attention is paid to this subject.

8.2.4 Conclusion

From the previous Sections it can be concluded that when a core of sand is applied the applied slope should be flatter or equal to 1:3.5 and that for a core of sand cement stone blocks the slope must be flatter or equal to 1:1.5 to ensure macro stability of the structures.
However, these values can not yet be regarded being design parameters. From the next Section it can be seen that the required stone size for the protection layer is also relative to the slope of the structure. Therefore here the final slope is not determined yet. This will be done when also the slope protection is discussed. This is done in the next Section.

8.3 Slope protection

8.3.1 Introduction

In this Section the required dimensions to resist both current and wave attack are calculated for all the slope protection materials. For these calculations a distinction is made for the various loads. Consequently the applied sizes are determined thereby taking in account the slope zones as determined in Section 8.1.3. Finally the criteria for the applied geotextiles are determined.

8.3.2 Resistance against current attack

8.3.2.1 Boulders and concrete blocks

For the determination of the required block size for slopes under current attack various formulae are available. The more refined the input data, the more exact the result.
As for the location no exact data are available for the hydraulic loads (see also Annex C) for this project a design formula is chosen which is in accordance to the data available. The formula contains factors which take account for the fact that not the exact velocity profile, but only the maximum mean verlocity in a vertical, is known. This formula, which is a slight simplification of Pilarczyk's formula (see ref [25]), yields:

$$\Delta D_n = \frac{2}{\Psi_{cr} B_1^2} \frac{K_h}{K_s} \frac{u^2}{2g} \qquad (4)$$

Where:

- Δ = relative density of the protection material (-),
- D_n = required nominal diameter of the protection material (m),
- Ψ_{cr} = Shields's critical shear stress parameter (-),
- B_1 = stability coefficient;
  - B_1 = 8 - 10 minor turbulence, uniform flow,
  - B_1 = 7 - 8 normal turbulence of rivers and channels,
  - B_1 = 5 - 6 major turbulence including local disturbances and constrictions, also outer bends of rivers[6].
- u = mean velocity in a vertical (m/s),
- g = gravity acceleration (m/s^2),
- K_h = depth factor (-); Here $K_h = (h/D_n)^{-0.2}$, which is the depth factor for a not fully developed velocity profile,

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[6]: In fact this factor makes the difference between the formula applied here and Pilarczyk's formula. Dividing these equations results in: $B_1 = \sqrt{\frac{2}{0.03\ \phi\ K_t}}$, where K_t is a turbulence factor (2/3 for low turbulence, uniform flow; 1.0 for normal turbulence and 1.8 for high turbulence, local disturbances and outer bends of rivers). ϕ is a stability factor for the protection material which can vary from 0.50 for continuous protections or mattresses to 1.25 for exposed edges of loosely units.

K_s = slope factor (-); for a slope perpendicular to the flow direction: $K_s = (1-\sin^2(\alpha)/\sin^2(\Theta))^{0.5}$. For a slope in the flow direction $K_s = \sin(\Theta-\alpha)/\sin(\alpha)^7$.

The simplification (see footnote 6) lies in the fact that here no account is taken for the stability of the slope protection material and the influence of turbulence with separated parameters as these would require detailed information for instance from model tests. A rough estimate could have been made for these parameters but this would introduce some fake refinement in the calculations. Using the formula above results in a reasonably high estimate for the combination of both these factors thereby also taking the fact into account that only the maximum mean velocity is known and not the local maximum current velocity.

Table II.10:Calculations for current attack on slope protection (1).

Resistance against current attack (Pilarczyk's formula)
Case 1: Boulders on a fascine matress

- slope angle 1 : 3.5 - slope perpendicular to the flow		
Parameter		Value
Depth	(m)	6
Velocity	(m/s)	1.6
Slope	1:m	3.5
Shields parameter	phi cr	0.030
Stab coef	B1	5
Angle of repose	Deg.	32
Rel. dens	Delta	1.6
Estim. Dn	(m)	0.20
Results:		
Slope angle	Deg.	15.95
Slope red. fact	Ks (-)	0.86
Depth factor	Kh (-)	0.51
Dn required	(m)	0.13
D50	(m)	0.15
M50	(kg)	5.6

- slope angle 1 : 3.5 - slope in flow direction		
Parameter		Value
Depth	(m)	8
Velocity	(m/s)	1.6
Slope	1:m	3.5
Shields parameter	phi cr	0.030
Stab coef	B1	5
Angle of repose	Deg.	33
Rel. dens	Delta	1.6
Estim. Dn	(m)	0.25
Results:		
Slope angle	Deg.	15.95
Slope red. fact	Ks (-)	0.54
Depth factor	Kh (-)	0.50
Dn required	(m)	0.20
D50	(m)	0.24
M50	(kg)	21.5

The calculations are made with the following considerations:

- The depth is taken as the depth of the toe of the structure below PWD. As the water level is always above this 0.00 PWD level this is a conservative estimate. This is done as a conservative estimate results in a higher D_n. The depth is taken into account via the depth factor K_h where a smaller depth results in a larger D_n.
- The velocity is taken at 1.6 m/s (see also Annex C). No extra safety coefficient is introduced as this formula already takes account for local deviations from the flow velocity.
- For boulders Shields' parameter is set at 0.030 which is the value for "no movement".

[7] Originally this slope factor applies to flow over a sill; at the downstream side the flow direction is along the slope thereby decreasing the stability of the protection material. Within this project this situation holds for the slopes at the downstream side near the head of the groyne; here a velocity, as applied in the calculations, can occur being directed downward along the slope.

Table II.11: Calculations for current attack on slope protection (2).

Resistance against current attack (Pilarczyk's formula)

Case 2: Concrete blocks on a core of artificial sand cement stone

- slope angle 1 : 1.5 - slope perpendicular to the flow		
Parameter		Value
Depth	(m)	15
Velocity	(m/s)	1.6
Slope	1:m	1.5
Shields parameter	phi cr	0.025
Stab coef	B1	5
Angle of repose	Deg.	41
Rel. dens	Delta	1.3
Estim. Dn	(m)	0.35
Results:		
Slope angle	Deg.	33.69
Slope red. fact.	Ks (-)	0.53
Depth factor	Kh (-)	0.47
Dn required	(m)	0.28
D50	(m)	0.28
M50	(kg)	52.7

- slope angle 1 : 1.5 - slope in flow direction		
Parameter		Value
Depth	(m)	8
Velocity	(m/s)	1.6
Slope	1:m	1.5
Shields parameter	phi cr	0.025
Stab coef	B1	5
Angle of repose	Deg.	42
Rel. dens	Delta	1.3
Estim. Dn	(m)	1.00
Results:		
Slope angle	Deg.	33.69
Slope red. fact	Ks (-)	0.22
Depth factor	Kh (-)	0.66
Dn required	(m)	0.98
D50	(m)	0.98
M50	(kg)	2178.3

- slope 1 : 2 - slope perpendicular to the flow		
Parameter		Value
Depth	(m)	15
Velocity	(m/s)	1.6
Slope	1:m	2.0
Shields parameter	phi cr	0.025
Stab coef	B1	5
Angle of repose	Deg.	41
Rel. dens	Delta	1.3
Estim. Dn	(m)	0.25
Results:		
Slope angle	Deg.	26.57
Slope red. fact.	Ks (-)	0.73
Depth factor	Kh (-)	0.44
Dn required	(m)	0.19
D50	(m)	0.19
M50	(kg)	16.7

- slope 1 : 2 - slope in flow direction		
Parameter		Value
Depth	(m)	8
Velocity	(m/s)	1.6
Slope	1:m	2.0
Shields parameter	phi cr	0.025
Stab coef	B1	5
Angle of repose	Deg.	42
Rel. dens	Delta	1.3
Estim. Dn	(m)	0.50
Results:		
Slope angle	Deg.	26.57
Slope red. fact	Ks (-)	0.40
Depth factor	Kh (-)	0.57
Dn required	(m)	0.46
D50	(m)	0.46
M50	(kg)	230.1

However, as the uniform grading of CC-blocks introduces the risk of layering which results in a chain reaction when one block lifts out here the criterion of "no movement" must be set a little more tight which results in a value of 0.025 for Shields' parameter.

- The stability coefficient, B_1, is set at 5. With the river defence works being located at an outer bend and also inducing some constriction this value seems to be appropriate.
- As the angle of repose is relative to the stone size used, the calculations are iterative. The angle of repose is taken into account via the slope factor and is derived from Figure II.11 where for the boulders the line "rounded" is used and for the cement blocks the line "angular". For the final calculation the angle of repose is chosen in such a way that it is not overestimated.

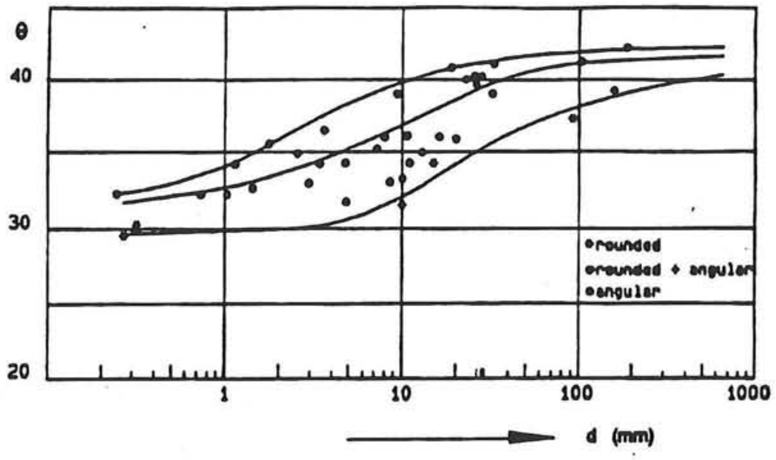


Figure II.11: Angle of repose for non-cohesive materials.

- The relative density, Δ, is taken at 1.6 for boulders and 1.3 for concrete. The value of 1.6 follows from BWDB specifications while the value of 1.3 is taken as a safe (low) estimate.
- The slope reduction factor is depending on the slope direction relative to the flow direction. With the slope in the flow direction the stability of protection material for a downward directed flow is decreased. This is expressed via the value for K_s which then is the lowest and results in the largest stone sizes required (see formula).
- The estimated D_n, as used in the depth factor, is taken larger than the finally determined D_n. As a larger estimate also results in a larger calculated D_n this is a safe estimate.
- The finally required stone diameter must be expressed in D_{50}. For boulders $D_{50} = 1.18\ D_n$ (rounded shape) while for the concrete blocks $D_{50} = D_n$ (cubic shape).

The results of these computations are presented in Table II.10 and II.11.

8.3.2.2 Open stone asphalt

The maximum velocity with respect to surface erosion of open stone asphalt is about 6 to 7 m/s (see e.g. ref [25]). With the maximum mean velocity here being 1.6 m/s it can be concluded that with respect to the resistance against current attack of open stone asphalt the margin in between is wide enough, also to be able to deal with turbulence aspects.

8.3.3 Resistance against wave attack

8.3.3.1 Boulders and concrete blocks

Here use is made of the following formula which is valid for breaking ("plunging") waves. It takes into account the duration of wave attack, the accepted damage level and the permeability of the underlying layers. It is known as a specific form of Van de Meer's formula (see ref. [26]) and reads as:

$$\frac{H_s}{\Delta D_n} = 6.2\ P^{0.18} \left(\frac{0.014S}{1.3(1-EXP(-0.0003N))}\right)^{0.2} \frac{1}{\sqrt{\xi}} \tag{5}$$

where:

- H_s = significant wave height (m),
- Δ = relative density of the protection material (-),
- D_n = required nominal diameter of the protection material (m),
- P = relative permeability of the protective layer to the core material (-), $P = 0.1$ for the practically impermeable core (sand or clay body) and $P = 0.6$ for the permeable (granular) core without a filter in between core and slope protection,
- S = damage level (-), a physical description for S is the number of cubical stones with a side of 1 x D_n, eroded over a width 1 x D_n. The "no damage" criterion is taken generally to be when S is between 1 and 3 stones eroded.
- N = number of waves (-), for this specific form of Van de Meer's formula N must be smaller than 1000 or larger than 7000,
- ξ = breaker index (-), defined as $\tan(\alpha)/\sqrt{(2\pi H_s/gT_z^2)}$, for the validity of the formula above ξ should be smaller than ξ_m,
- ξ_m = maximal value for the breaker index ξ, for which the waves are still breaking (-), this index, which in the calculations is called the transition index, is given from the following formula:

$$\xi_m = (6.2\ P^{0.31} \sqrt{\tan(\alpha)})^{\frac{1}{P+0.5}} \tag{6}$$

where the parameters are as given above.

The calculations are elaborated with the following considerations in mind:

- The wave height, as used in the formula, is derived in Annex C. The design wave with a return period of 100 years has a height of 0.96 m. The wave parameters are derived for a storm with a duration of 900 sec (the so called "Nor-Wester" thunderstorm squalls, see Annex

Table II.12: Calculations for wave attack on slope protection.

Resistance against wave attack (Van der Meer's formula for N<1000)
Slope protection by boulders or concrete blocks

Boulders		
Slope 1 : 3.5		
Parameter		**Value**
Wave height	(m)	0.96
Wave period	(s)	3.46
Storm duration	(s)	900
Slope	1:m	3.5
Damage number	(%)	2
Rel. density	(-)	1.6
Rel. permeability	(-)	0.1
Results:		
Slope angle	Deg.	15.95
Number of waves	(-)	260
Breaker index	(-)	1.26
Transition index	(-)	2.24
Dn required	(m)	0.21
D50	(m)	0.25
M50	(kg)	24.5

Concrete blocks		
Slope 1 : 1.5		
Parameter		**Value**
Wave height	(m)	0.96
Wave period	(s)	3.46
Storm duration	(s)	900
Slope	1:m	1.5
Damage number	(%)	1
Rel. density	(-)	1.3
Rel. permeability	(-)	0.6
Results:		
Slope angle	Deg.	33.69
Number of waves	(-)	260
Breaker index	(-)	2.94
Transition index	(-)	3.78
Dn required	(m)	0.33
D50	(m)	0.33
M50	(kg)	82.9

Concrete blocks		
Slope 1 : 2		
Parameter		**Value**
Wave height	(m)	0.96
Wave period	(s)	3.46
Storm duration	(s)	900
Slope	1:m	2
Damage number	(%)	1
Rel. density	(-)	1.3
Rel. permeability	(-)	0.6
Results:		
Slope angle	Deg.	26.57
Number of waves	(-)	260
Breaker index	(-)	2.21
Transition index	(-)	3.32
Dn required	(m)	0.29
D50	(m)	0.29
M50	(kg)	53.8

C). As the computations result in two wave periods (depending on the model used) here the maximum wave period, which results in the highest load, of 3.46 sec. is used.

- For the boulders S is set at 2% which is the normal "no damage" criterion. But, similar to the situation with currents, for the concrete blocks the damage criterion is set more strict to deal with the risk of the collapse of layered protection material which is more progressive. For the CC-blocks protection S is taken being 1%.

Furthermore parameters are taken with the same considerations as for current attack.

The calculations are presented in Table II.12. It can be seen that for all calculations $\xi < \xi_m$ so the same formula is valid for each situation.

8.3.3.2 Open stone asphalt

The formulae available for the determination of the required thickness of a cover layer of bounded aggregates with respect to wave attack are quite simple. The best usable formula is a modification of Van de Meer's formula and reads as:

$$\frac{H_s}{\Delta D} = \Psi_u \frac{\phi}{\sqrt{\xi_z}} \qquad (7)$$

Where:

- H_s = significant wave height (m),
- Δ = relative density of the protection material (-),
- D = required thickness of the layer of open stone asphalt (m),
- Ψ_u = upgrading factor (-), due to the interconnection of the protective layer, for open stone asphalt this parameter can be set at 5.0,
- ϕ = stability factor of the protective material on its sub base (-), for open stone asphalt on a geotextile this parameter can be set at 1.0,
- ξ = breaker index (-), defined as $\tan(\alpha)/\sqrt{(2\pi H_s/gT_z^2)}$.

The hydraulic parameters (H_s, ξ), as used in this formula, are similar to the ones used in Van der Meer's formula above. The factors are used as described above and the relative density, Δ, of open stone asphalt is taken at 1.0 here.

For a slope of 1:3.5 this results in a required thickness of open stone asphalt of 0.22 m.

8.3.4 Slope compositions

8.3.4.1 Required sizes of slope protection material

- Boulders and concrete blocks

The results from the calculations for current and wave attack are summarized in the Table below:

***Table II.13**: Required sizes for units of slope protection material.*

Load / Case	Sand core + Boulders on geotextile		Sand cement stone core + Concrete blocks			
Slope	1 : 3.5		1 : 1.5		1 : 2	
Orientation	Slope ∥ flow	Slope ⊥ flow	Slope ∥ flow	Slope ⊥ flow	Slope ∥ flow	Slope ⊥ flow
D_n Currents	0.20 m	0.13 m^A	0.98 m	0.28 m	0.46 m	0.19 m
D_n Waves	0.21 m		0.33 m		0.29 m	

Note A) For the guide bank structure (where the depth is larger) the required D_n is a little smaller namely 0.11 m. In view of the very small difference this is further not taken into account. The required stone size here is governed by construction demands. With the sizes of the fascines being approximately 0.30 m and assuming a layer thickness of $2{*}D_n$ the more practical size of D_n is 0.15 m. must be applied here.

In the Table the determining loads are marked with an asterisk (*). It can be concluded that for steep slopes parallel to the flow direction (steeper than or equal to 1:2, when a core of sand cement stone is applied) currents are the determining loads. As the zone of current attack is along the entire structure (see Section 8.1.3) and along the whole slope CC-blocks are applied the whole slope will have to be covered with the block sizes required for wave attack.

For flatter slopes (1:3.5, when a sand core is applied), or if the slope is directed perpendicular to the flow, wave attack requires larger stone sizes than current attack. Therefore in this case a distinction must be made between the zones of wave and current attack.

From this Table it can be seen that when a slope of 1:1.5 is applied which is directed parallel with the flow (for the groynes) a very large stone size (0.98 m) is required. Though this slope is stable on a macro scale and demands the fewest core material it is not advisable to apply it as the construction and handling of these sizes of CC-blocks will cause major problems. Therefore for the construction of short groynes with a core of sand cement stone blocks a slope of 1:2 is advised. The sizes of the required CC-blocks here can be handled by manual labour, using special tongs, but also heavy enough to prevent theft in the final situation. For the guide bank structure with a core of sand cement stone a slope of 1:1.5 can be applied and for all structures with a core of sand (bank revetment, long groynes and guide bank) a slope of 1:3.5 is applied.

- Open stone asphalt

From the foregoing calculations it can be concluded that a thickness of 0.22 m will be sufficient to withstand all loads. Therefore this thickness is applied.

8.3.4.2 Applied sizes of slope protection material

From the conclusions in the previous Section here the applied slope compositions are determined. They are presented in Table II.14:

***Table II.14**: Applied sizes for units of slope protection material.*

<table>
<tr><td>Core material</td><td colspan="2">Sand/hydraulic fill with cont. bunds</td><td colspan="2">Sand cement stone blocks</td></tr>
<tr><td>Structure type
Slope zone[8]</td><td>Guide bank
or revetment
(slope 1:3.5)^A</td><td>Groynes
(slope 1:3.5)^A</td><td>Guide bank
(slope 1:1.5)</td><td>Groynes
(slope 1:2)</td></tr>
<tr><td>I</td><td rowspan="2">Boulders on a
fascine mattress
D_n = 0.15 m.</td><td rowspan="2">Boulders on a
fascine mattress
D_n = 0.15 and 0.20 m.</td><td rowspan="3">CC-blocks

0.30 m^3</td><td>CC-blocks 0.20 m^3
and 0.30 m^3</td></tr>
<tr><td>II</td><td rowspan="2">CC-blocks 0.30 m^3
and 0.45 m^3</td></tr>
<tr><td>III</td><td colspan="2">Open stone asphalt (thick 0.22 m.)</td></tr>
</table>

[8] For the definition of the slope zones see Section 8.1.3.

Note A) The transition between these two types of protection material is done by a transition zone of bitumen grouted boulders. A detail is given in the drawings of the alternatives (Section 8.5).

The variety in types of material is kept limited here and difficulties during construction are avoided as much as possible. Therefore no separate stone diameter is applied in slope zone II, which is very small (only 1.50 m in height).

The applied block sizes, given in Table II.14, are determined for the total alternatives, as determined in Section 7.6. The considerations were the following:

- **For alternative I** (Guide bank + river bank revetment, both with a sand core)

For the guide bank and the river bank revetment both the direction is perpendicular to the flow. Therefore in slope zone I and II boulders with a D_n of 0.15 m. are applied. As in slope zone II also wave influences are expected in fact a D_n of 0.21 m is required (see Table II.13). However, as slope zone II is very small (only 1.50 m in height) no distinction is made here.
From van de Meer's formula it can be seen that applying a D_n of 0.15 instead of the required 0.21 m results in a damage factor (S) of 6.0 % instead of the required 2.0%. This is not within the limits of "no damage" but it will not cause damage to the structure itself. As within the computations the effect of the bitumen grouting is not included these damage numbers can be expected to be a worst expectation. As the probability of occurrence for the critical situation (i.e. a storm with a 100 years return period and a water level with a 90% exceedance in April at the same time) is very low this is regarded acceptable. A damage factor below 10% damage will not lead to failure of the structure itself. If the critical situation ever occurs quick maintenance will have to be done.

- **For alternative II** (Guide bank + long groynes, both with a sand core)

Here for the guide bank the same considerations hold as the ones discussed for alternative I.
For the groynes however, their direction to the flow is perpendicular. Therefore at the head and the downstream side near the head a downward directed flow along the slope can occur. At the head of the groyne also an increased turbulence can be expected.
Therefore at these parts of the structure in slope zone I and II boulders with a D_n of 0.20 m are applied. The lenght at the downstream side from the head where this larger boulder size is applied is taken 10% of the entire length of the groyne (so 20 m). In fact this is not required for slope zone I but as it is known that the head of a groyne is always the part of the structure where attack is the most severe this is done as a safety measure. In slope zone III, where open stone asphalt no special measures are taken as open stone asphalt can be assumed to be safe and stable for currents up to 6 à 7 m/s.
Along the structure connecting the groyne head with the river bank again no distinction is made for slope zone II. This results in the same damage factor of about 6% (see the discussion above for alternative I).

- **For alternative III** (Guide bank + short groynes, both with a core of sand cement stone blocks)

For the guide bank with a slope of 1:1.5 along the entire slope CC-blocks with a D_n of 0.30 m are applied. Here also no distinction is made for slope zones II and III where in fact a D_n of 0.33 m is required. From van de Meer's formula it can be seen that the damage factor (S) will be around 1.6% instead of 1.0%. As this damage factor is still within the limits of "no damage" (1-3%) no problems are expected.

For the groynes, with a slope of 1:2, here also slope zone III is vulnerable (in contrast with alternative II) Therefore here a block size of 0.45 m^3 is applied at the head and the downstream slope near the head in slope zone II and III. Here the length along the groyne body is also taken 10% (so 10 m). In slope zone I, where higher turbulence can be expected a block size of 0.30 m^3 is applied instead of the 0.20 m^3 size which is strictly required.

To be able to resist the wave loads a size of 0.30 m^3 is sufficient for the rest of the structure for slope zone II and III. This size is also applied for slope protection of the guide bank. Therefore the variety in sizes is limited to a total number of three.
Along the body in slope zone I (the zone of current attack were construction has to be done in the wet) block sizes of 0.20 m^3 are applied.

Safety against theft is provided here as the CC-blocks which are the easiest to grab (in slope zone II and III) have a minimum weight of about 55 kg (for a D_n of 0.30 m, see Table II.11).

Further details are given in Section 8.5, where the final designs are presented.

- Applied gradings

The CC-blocks are uniformly graded (artificially produced) and the boulders are normal graded. This yields:

- a grading of 0.15 to 0.25 m. for a D_n of 0.20 m. and
- a grading of 0.10 to 0.20 m. for a D_n of 0.15 m.

8.3.4.3 Quantities of slope protection material

- Thickness of slope protection layers

The thickness of the slope protection layers for dumped material follows from the semi empirical formula:

$$t = mK_{\Delta}d_n \tag{8}$$

Where:

t	=	thickness of layer (m),
m	=	number of layers (-), usually m =2, sometimes 1 or 3,
K_{Δ}	=	empirical layer coefficient (-) depending on the applied protection units,
d_n	=	the nominal diameter of the protection unit (m), as calculated above.

For randomly placed (dumped) concrete cubes K_{Δ} is about 1.10 and for rough quarry stone (rock) K_{Δ} is between 1.00 and 1.15 while for smooth quarry stone (boulders) K_{Δ} is about 1.02. For all structure types two layers will be applied.
If a fascine mattress is applied the minimal layer thickness is about 0.30 m, in view of the thickness of the fascines.

The layer thickness for open stone asphalt is determined already in Section 8.3.4.1.

The applied layer thickness for all slope protection materials is presented in the following Table:

***Table II.15:** Thickness of slope protection layers.*

Material	Slope zone	d_n (m)	t (m)
CC-blocks	I	0.20	0.44
	I, II, III	0.30	0.66
	II, III	0.45	0.99
Boulders	I,II	0.15	0.31
	I,II	0.20	0.41
Open stone asphalt	III	0.22	0.22

- Quantities

For boulders the needed quantity is expressed in a volume which follows from integrating the total surface of the slope protection and the thickness of the slope protection layers as determined with the formulae mentioned above. Also for open stone asphalt the surface and the thickness are integrated.

For concrete blocks the needed quantity can also be expressed in the number of units which follows from the following empirical formula:

$$C = \frac{mK_{\Delta}(1-n)}{d_n^2} \tag{9}$$

where:

- C = the number of slope protection units per unit area of slope protection ($1/m^2$),
- m = number of layers (-), usually m =2, sometimes 1 or 3,
- K_{Δ} = empirical layer coefficient (-) depending on the applied protection units,
- n = porosity of the slope protection layer expressed as a decimal (-),
- d_n = the nominal diameter of the protection unit (m), as calculated above,

For randomly placed (dumped) concrete cubes the porosity is about 47%. Other parameters are as mentioned above.

The total required number of units follows from the integration of the slope protection area, multiplied with C.

8.3.5 Filters

8.3.5.1 Introduction

In Section 7.3.4 it is remarked already that when a sand core is applied a filter is needed in between the slope protection and the core material. In the material evaluation it is already decided to apply geotextiles. In this Section these demands are specified further and a conclusion is drawn about the geotextiles to be applied.

The demands which geotextiles must fulfil are mainly governed by the type of ground water flow pattern. For this particular situation the ground water flow pattern can be described as follows:

- (Ground water) flow pattern

The ground water flow pattern for the protected slopes (1:3.5) is a (semi-) stationary laminar flow and can be described with Darcy's law. Within the S.T.S. it was already derived that the maximal ground water flow gradient perpendicular to the slope surface (i_{max}) for this situation, which follows from a square-grid approximation of the ground water flow situation can be estimated to be around 0.05.

From the general filter requirements, as discussed in Section 7.3.4.1, the demands which must be fulfilled by a geotextile and the subsoil, can be derived. These demands concern:

- internal stability of the subsoil,
- sand tightness of the filter,
- permeability higher than the subsoil,
- strength to resist forces due to settlements/deformations and during construction,
- resistance against sliding and/or uplifting,

Also additional demands as climatological resistance, durability etc. can be recognized. In the next Section the demands are discussed and finally a conclusion is drawn for the type of geotextile to be applied.

8.3.5.2 Filter demands

- Internal stability of the subsoil

The subsoil can be assumed to be internally stable if the ratio $d_{60}/d_{10} < 10$. From the particle size distribution, as presented in Annex E, it can be derived for both the layer types I and II that this ratio is about 5 (layer type I and II are the only types which are present at the location of the Meghna Bridge, see Annex E). Therefore it can be concluded that this requirement is fulfilled.

- Sandtightness

A distinction can be made between geometrically closed and open filters. Closed filters are sandtight independent of the (ground water) flow pattern while open filters are only stable below a certain critical flow velocity. As the exact ground water flow pattern is not known for this situation here a closed filter type will be applied. For non cohesive soils (layer I or II type) the demands for the two types read as follows:

$O_{90\ (filter)}/D_{90\ (soil)}$ < 1.0 for wovens and
$O_{90\ (filter)}/D_{90\ (soil)}$ < 1.8 for non-wovens.

To assure the permeability demands the O_{90} should be as close as possible to these 1.0 or 1.8 figures respectively.

Taking the characteristic $D_{90\ (soil)}$ as 0.08 mm (which was determined in the S.T.S. for the location of the Meghna Bridge) this demand results in:

$O_{90\ (filter)}$ is about 0.08 mm for wovens and
$O_{90\ (filter)}$ is about 0.14 mm for non wovens.

This requirement can only be met by a non woven geotextile.

- Permeability

As mentioned already before the permeability of a filter must be significantly larger than that of the sub-soil to prevent pressure building. A rule of thumb is the demand that $k_{geotextile}$ = 5 times k_{soil}. This demand holds for both the geotextile types. Within the S.T.S. the permeability of the soil at the location of the Meghna Bridge was determined to be $1.576*10^{-7}$ m/s. This results in a required permeability for the geotextiles of about $0.8*10^{-6}$ m/s.

- Strength

The geotextile must be able to resist the forces to which it is exposed during construction. As for this project the main purpose of the geotextiles is the prevention of erosion of the soil and the drainage of the ground water level and not armouring against deformations loads during construction will be determining. However, when determining the type of geotextile, account has to be taken for its flexibility to follow possible settlements.

- Stability against sliding

The stability against sliding of the geotextile is governed by the applied slope angle, the ground water gradient perpendicular to the slope and the interface friction angle. As the slope angle is already determined by the macro slope stability with the demand of the steepest possible slope (which results in the smallest dredging volume) here the only parameter left to be determined is the interface friction angle. The required friction angle follows from:

$$\tan(\beta) \geq \frac{n \tan(\alpha)}{(1-i)} \qquad \textbf{(10)}$$

where:

ß	= required friction angle at interface soil-geotextile (°),
n	= safety factor (-),
α	= slope angle (°),
i	= ground water gradient perpendicular on the slope (-).

The critical situation, as expressed in the formula above, only yields at the water line where the maximum ground water gradient, perpendicular to the slope, occurs. As sliding of the geotextile will be prevented by the upper part (if the strength is sufficient) the requirement as discussed here is not very strict.
Taking α = 15.96° (slope 1:3.5), n = 1.5 (see Annex E), and i = 0.05 results in a required ß of 25°.

8.3.5.3 Applied geotextile type

For the finally determined geotextile type here no distinction is made on which soil type it is applied. Such a distinction would only increase the risk of errors during construction.

The demands as listed above can only be met by a combination of both types of geotextiles (woven and non-woven). Therefore a composite geotextile is required which consists of a. combination of woven, for the strength, and non-woven, for the sand tightness. The interface geotextile-geotextile will be vulnerable to sliding, therefore some kind of interconnection between the two geotextiles will have to be established.

Here the same combination of geotextiles can be applied as done within the S.T.S. for the designs of Bhairab Bazar. These geotextile types were specified as follows:

***Table II.16**: Specifications geotextile type I (woven).*

Item	Specification
Material	100% woven polypropylene
Effective pore size	$200*10^{-3} < O_{90} < 300*10^{-3}$ m.
Permitivitty	$< 0.1\ s^{-1}$
Strength (wrap and weft)	70 kN/m
Weight	450 gr/m^2

***Table II.17**: Specifications geotextile type II (non woven).*

Item	Specification
Material	100% non woven polypropylene
Effective pore size	$O_{90} < 0.125*10^{-3}$ m, $O_{50} < 0.075*10^{-3}$ m.
Permitivitty	$> 0.1\ s^{-1}$
Strength	> 70 Kn/m
Weight	> 200 gr/m^2
Grab strength	> 900 N

The composite geotextile will be placed underneath the open stone asphalt layer and integrated into the fascine mattress. The woven is laid at the bottom and the non woven on top. Therefore the woven geotextile will have to fulfil the requirement of $\beta > 25°$.

When a core of sand cement stone blocks is applied no filter is needed in the final situation. However, during construction a filter under the toe is needed to prevent heavy scour before construction of the apron is completed. Also in later stages when the apron material is replaced such a filter ensures the safety of the structure.
Therefore during construction of a guide bank or groyne from sand cement stone blocks geotextiles (only type I) are placed under the toe. Only type I geotextile is applied here as the requirements are not so strict as for the situation described above. They are placed with a safety margin of two meters (see drawings in Section 8.5). It is not preferable to extend them further under the apron section as this would disturb the uniform launching of apron material and could induce erosion of bed material under the geotextile. For further details concerning the falling apron see the next Section.

8.4 Falling apron

8.4.1 General concept

The main task of the falling apron section is to provide safety of the structures against the dangers induced by the maximum scour depths in front of it. The deep scour holes must be kept at a certain distance from the toe of the structure, which is levelled at the present river bed. To be able to do so an amount of graded material is dumped in front of the toe of the structure (the so called apron). The apron material will (partly) start sliding down the slope of a scour hole if it reaches the toe. The sliding material will cover the scoured hole partially thereby stopping (slowing down) the scour hole to shift further to the structure's toe. If the apron material is widely graded, a natural filter will develop; from the top the finer fractions are flushed out and the larger sizes, which are able to resist the hydraulic loads remain. Below these larger sizes the finer fractions are more or less protected and these fractions will more or less form a granular filter.

From the working concept of a falling apron, as described above, it is clear that (parts of) the covered scour slope will develop at about the natural slope of the soil material. This results in a safety factor of $n = 1$. From various studies and handbooks where measurings at falling aprons were presented it can be derived that the slope after sliding will be around 1:2, the natural slope of the sub soil.

To minimize the risk of malperformance of the apron the following requirements should be taken account for:

- The angle of repose of the underlying material (i.e. the river bed) should be flatter than that of the apron material to induce the sliding of the apron blocks (if not, the whole apron would slide along the slope).
- The construction base must be similar all along the apron to induce a uniform sliding. So alternating clay layers must be avoided.
- The apron material should consist a sufficient quantity of material which is able to resist the (flow) forces on a slope of 1:2.
- The apron material must be widely graded. This will enable the development of a filter layer which can retain the underlying soil layers. Another advantage of a wide grading is that part of the apron material slides more easily. This results in an earlier induced covering process which results in flatter slopes in front of the structure.

The first two requirements are easy to match. The base material is soil of a layer I type (see Annex E). This material is very uniform and does not contain any clay. Therefore the risk for the occurrence of alternating layers is minimal. Using the materials as proposed in Section 7.4.2 for the apron section it is also ensured that the angle of repose of the underlying material (25° to 27°) is always less than the angle of repose of the apron material (32° to 37°, see the calculations in the next Section).

The other requirements are discussed in the next Sections.

A falling apron can be considered being a "self constructing" filter of graded material of which the larger sizes are resistant to the loads induced by the currents. The apron material is dumped in a wedge shape; at the toe of the structure the thickness of the apron is equal to the thickness of the cover layer of the structure. This thickness increases over a length L_f in such a way that the required quantity of apron material is stocked. In order to store as much material as possible at the river side of the apron the slope at the river side is as steep as possible (1:1). The determination of the required quantity of apron material and the length L_f is discussed in Section 8.4.3. An example of an apron cross section is given in Figure II.12.

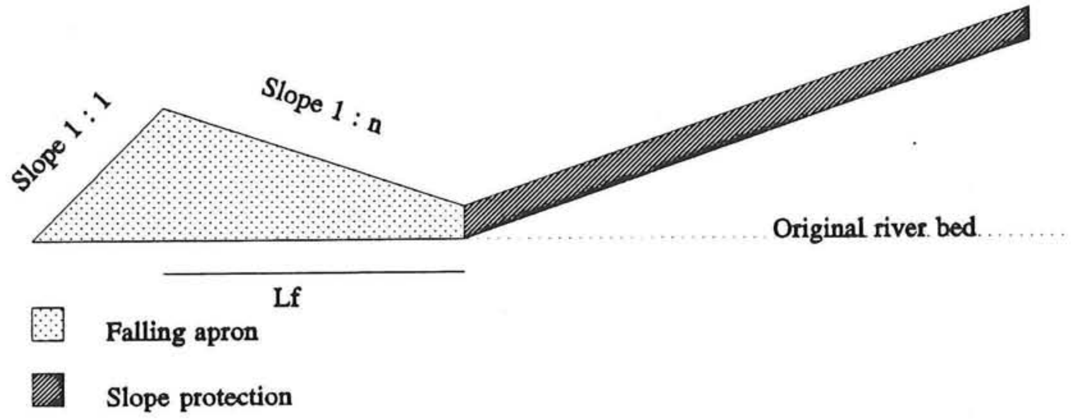


Figure II.12: Typical cross section for a falling apron.

As the working and behaviour of the material in a falling apron is not completely predictable it must be borne in mind that always maintenance will have to be done. After a period of about ten years it might be needed to dump a new amount of apron material. Depending on the hydraulic characteristics in the meantime however, this moment can also be earlier or later.

In the following Sections the required sizes, gradings and quantities for the proposed apron materials are determined.

8.4.2 Stone sizes and grading

Sizes required for current resistance

The large stone sizes, which in the final state (after sliding) must be able to resist the hydraulic (current) loads, are determined assuming the steepest slope to be 1:2. Use is made of Pilarczyk's formula (see Section 8.3.2.1) for resistance against current attack. The calculations are presented in Table II.18.

For this case in Pilarczyk's formula some parameters are taken different than for the slope protection calculations. These are the following:

Table II.18: Calculations for current attack on apron material.

Resistance against current attack (Pilarczyk's formula)

Case 3: Falling apron section of boulders (graded)

- slope 1 : 2
- slope perpendicular to the flow

Parameter		Value
Depth	(m)	10
Velocity	(m/s)	1.6
Slope	1:m	2
Shields parameter	phi cr	0.030
Stab coef	B1	6
Angle of repose	Deg.	32
Rel. dens	Delta	1.6
Estim. Dn	(m)	0.15
Results:		
Slope angle	Deg.	26.57
Slope red. fact.	Ks (-)	0.54
Depth factor	Kh (-)	0.43
Dn required	(m)	0.12
D50	(m)	0.14
M50	(kg)	4.7

Case 4: Falling apron section of CC-blocks

- slope 1 : 2
- slope perpendicular to the flow

Parameter		Value
Depth	(m)	10
Velocity	(m/s)	1.6
Slope	1:m	2
Shields parameter	phi cr	0.030
Stab coef	B1	6
Angle of repose	Deg.	37
Rel. dens	Delta	1.1
Estim. Dn	(m)	0.20
Results:		
Slope angle	Deg.	26.57
Slope red. fact.	Ks (-)	0.67
Depth factor	Kh (-)	0.46
Dn required	(m)	0.15
D50	(m)	0.15
M50	(kg)	7.1

- The depth is taken 10 m for all calculations. This is a safe approach. Taking a depth of 20 m results in required sizes which are about 10% smaller. As however also for the guide bank depths of about 10 m occur (at the heads) this depth is taken the same for all structure types.
- The slopes are taken 1:2. This is the steepest expected slope of an apron after sliding.
- The stability coefficient is taken 6 here. At the concerned depths the turbulence is expected to be smaller than for the slope protection calculations elaborated before. Also movement of some protection material is not extremely dangerous, it might even be preferable (see the discussion, concerning the requirements, above).
- Shields' parameter is set at 0.030.
- The angle of repose is derived from Figure II.11, here the line "rounded and angular" is applied as here the concerned material is CC-blocks with brick aggregates.

- Grading

The sizes, as determined above, are the sizes required to resist the hydraulic loads due to the current. However, also smaller sizes are required to provide material which can form a filter layer. The normal procedure is to take into account several safety factors which determine the required excess quantity. This results in a required layer thickness of $5 * D_n$ (D_n = the nominal stone diameter resistant against current attack) for the apron. An extra quantity of 25% is taken to be able to deal with losses due to material which does not reach its intended place. This results in a total amount of 6.25 D_n. As also lots of finer particles are required the D_n, as determined for current resistance, can be taken as the D_{65} in the overall grading for the apron consisting of boulders. This grading should be taken wide. The applied grading is given in the Table II.19:

Table II.19: Applied grading of apron material.

	D_n (D_{65} in grading) in meters	Grading in meters
Boulders	0.12	0.05 to 0.20

8.4.3 Material quantities

8.4.3.1 General

As already mentioned above material losses will occur during construction and due to erosion of non current resistant material when the apron material slides.

Also an extra quantity must be foreseen extra for outfanning in curved areas, from calculations a factor (f) of 1.3 follows.

This results in the following general formulae, with which the required quantities and dimensions of the apron can be determined:

$$T_f = 5\ D_n\ 1.25 \tag{11}$$

$$Q_f = D_s\ T_f\ \sqrt{5}\ f \tag{12}$$

$$L_f = 1.25\ D_s \tag{13}$$

where:

- T_f = Thickness of apron after sliding (m),
- D_n = Nominal diameter of current resistant stone (m),
- Q_f = Quantity of required apron material (m^3/lin.m)
- D_s = Design scour depth referred to toe level (m),
- f = Outfanning factor (-), f = 1.0 for straight reaches and 1.3 for curved areas,
- L_f = Length of falling apron (m), see Figure II.12.

In the next Sections the apron quantities are calculated for the cross section as presented in the drawings. Drawings are made for the cross section about 200 m. downstream of cross section U3 which in the alternatives is the location of the second long groyne (LG2) or the fourth short groyne (SG4) for the river bend fixation works. Also some characteristic apron quantities are calculated for the projected guide bank.

As the required quantities are relative to the scour depth to cover (D_s) which in its turn depends on the applied slope of the structure a distinction is made here for the various core materials.

8.4.3.2 Quantities for aprons of boulders

In this Section the slope of all structures, which determines the toe level, is 1:3.5. For the river bend fixation alternatives some characteristic values are given in the next Table:

Table II.20: Quantities for aprons of graded boulders in bend fixation alternatives I and II.

Cross section 200 m d/s U3:	Toe level (m PWD)	D_s (m)	T_f (m)	Q_f (m^3/lin m)	L_f (m)
Groyne with sand core (curved head)	- 10.0	14	0.75	30.50	17.50
Revetment	- 4.0	19	0.75	31.90	23.75

The design scour level follows from interpolation and is 24 m-PWD for the groynes here and 23 m-PWD for the revetment.

For the curved head of the groyne the factor f = 1.3 is taken into account. At the deepest part of the adjacent reach the apron quantity is calculated without this fanning factor but with another toe level. Details are given in the drawings presented in Section 8.5.

For the guide bank the toe level varies from 12 m-PWD at the river side of the up- and down-stream curves to 26 m-PWD in the middle of the guide bank, due to the scour hole present here. Apron sizes for both toe levels are given in the Table below.

Table II.21: Quantities for an apron of graded boulders for a guide bank (alternative I and II).

Guide bank (sand core):	Toe level (m PWD)	D_s (m)	T_f (m)	Q_f (m^3/lin m)	L_f (m)
Upstream curve	- 12.0	18	0.75	39.20	22.50
Deepest toe level	- 26.0	4	0.75	6.70	5.00
Downstream curve	- 13.0	17	0.75	37.10	21.25

Here also for the curved areas the factor f of 1.3 is taken into account. Therefore the required quantity at the adjacent straight reach of the guide bank is only 37.10/1.3 = 28.50 (m^3/lin m) and 39.20/1.3 = 30.20 (m^3/lin.m)

8.4.3.3 Quantities for aprons of CC-blocks with brick aggregates

In this Section the slope of the structure is 1:1.5 for the projected guide bank and 1:2 for the groynes.

For the river bend fixation alternative some characteristic values are given in the Table below.

Table II.22: Quantities for an apron of CC-blocks with brick aggregates for a groyne.

Cross section 200 m d/s of U3:	Toe level (m PWD)	D_s (m)	T_f (m)	Q_f (m^3/lin m)	L_f (m)
Groyne with sand cement stone core (curved head)	- 9.0	15	0.94	40.80	18.75

Here the same applies as for Table II.20.

For the guide bank the toe level varies from 12 m-PWD at the river side of the up- and down-stream curves to 18 m-PWD in the middle of the guide bank, due to the present scour hole here. Apron sizes for both toe levels are given in the Table below.

Table II.23: Quantities for an apron of CC-blocks with brick aggregates for the projected guide bank.

Guide bank:	Toe level (m PWD)	D_s (m)	T_f (m)	Q_f (m^3/lin m)	L_f (m)
Upstream curve	- 10.0	20	0.94	54.60	25.00
Deepest toe level	- 18.0	12	0.94	25.20	15.00
Downstream curve	- 14.0	16	0.94	43.70	20.00

Here also for the curved areas the factor f of 1.3 is taken into account. Therefore the required quantities at the adjacent straight reach of the guide bank are only 54.60/1.3 = 42.00 and 43.70/1.3 = 33.60 (m^3/lin m) respectively.

8.4.4 Reductions of apron quantities along structures

- Guide bank

Along the guide bank the apron quantities are linearly interpolated from the values for non curved areas, given above. At the curved heads the apron quantity is taken constant up ot the place where the curve exceeds 180° relative to the general crest direction of the guide bank. From this point the apron quantity is reduced to 0 at the end of the structure.

- Groynes

At the groyne heads the apron quantities are taken as given in the Table above. These values do take into account outfanning. Along the groyne body (the connection between groyne head and river bank) the quantity of apron material can be reduced as the expected scour depths decrease here in the direction of the river bank. Therefore the amount of material at the connection with the groyne head is determined and applied over 0.25 times of the length at the upstream side and 0.50 times the length along the downstream side.

- Revetment

It is not predictable where the deepest scour holes along the revetment will occur. Therefore no reductions (different than the ones mentioned in Section 8.1.1) along the projected river bank revetment can be applied. Between the various cross sections for which the scour levels are determined (see Section) the values are linearly interpolated. Here interpolation can be done in steps in order to facilitate construction.

8.5 Final alternatives

In the next Sections some additional details concerning the alternatives are discussed and subsequently the drawings are given. For each river bend fixation alternative a typical cross section is given for the location about 200 m. downstream of cross section U3. The location of this cross section is shown in Figure II.13. For each guide bank option two drawings are given: a general layout and one cross section of the structure at the deep scour hole.

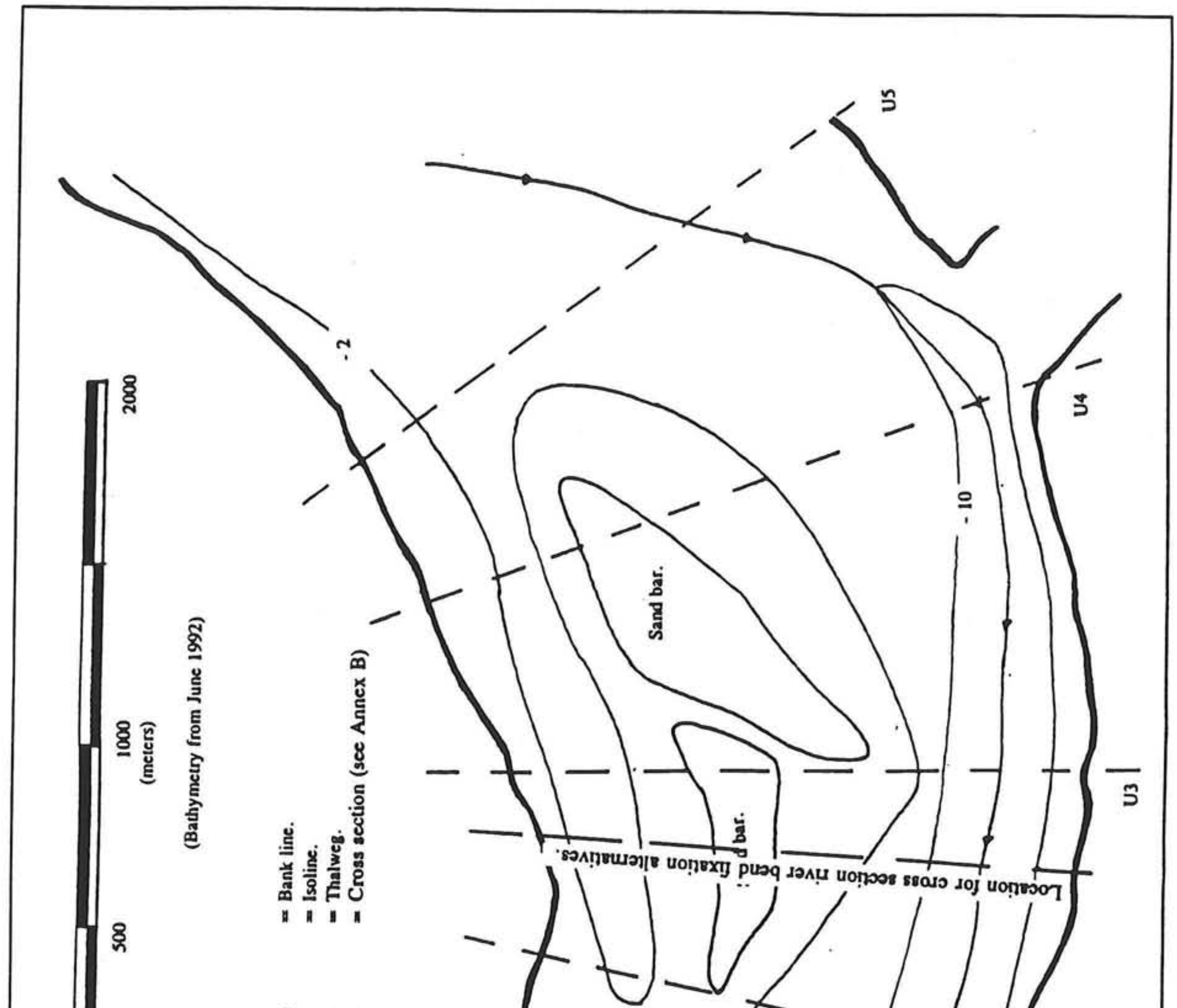


Figure II.13: Location of the typical cross sections for the river bend fixation alternatives.

8.5.1 Alternative I

- Guide bank

The required slope is 1:3.5. With the chosen alignment this results in a slope which reaches into (over) the deep scour hole. As construction down to the present bed level is the safest it is decided not to fill the deep scour hole by a hydraulic fill. Therefore the deepest toe level is 24 m-PWD.

The design scour level for the guide bank is taken at 30 m-PWD. At the upstream curve the design scour depth is gradually decreased to 25 m-PWD which is the design scour depth for river bank revetment at cross section U1 (the former Ferry Ghat).

Along the entire guide bank the crest level is set at 6.0 m+PWD. Where the guide bank reaches the present river bank line the height is gradually decreased to the bank level (4.0 m+PWD).

As part of the structures is constructed from land and in order to facilitate possible maintenance operations here a crest width of 6.0 m for all structures is applied.

Drawings of the guide bank for alternative I are given in Figure II.14 and II.15. A typical fascine mattress with boulders of 0.15 m. is given in figure II.16.

- River bank revetment

River bank revetment is required over a total reach of 3000 m. The side channel just downstream of cross section U2 is closed. A typical cross section and a detail of the transition between the open stone asphalt and the boulders on a fascine mattress is given in Figure II.17.

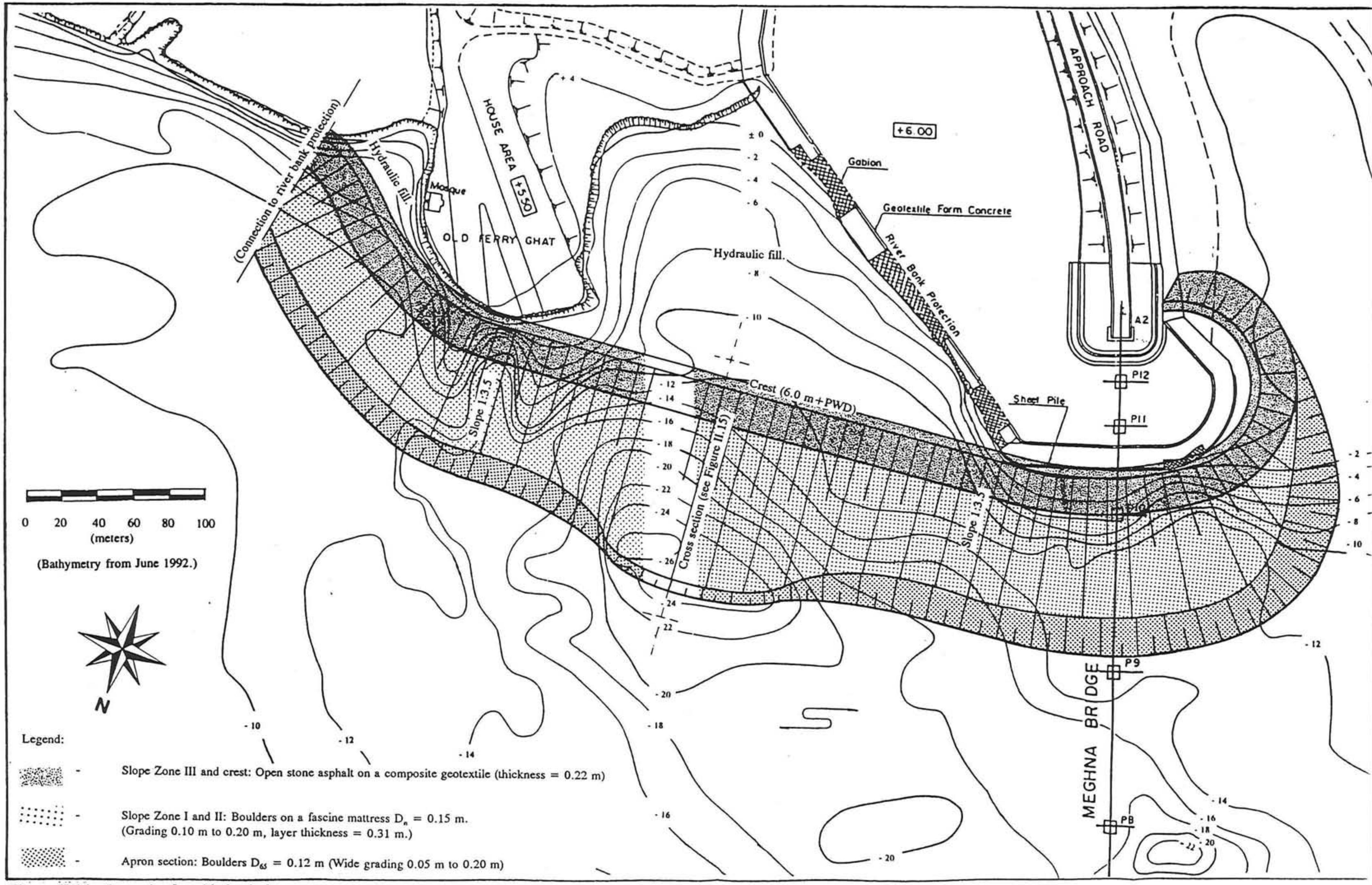


Figure II.14: Alternative I, guide bank; layout.

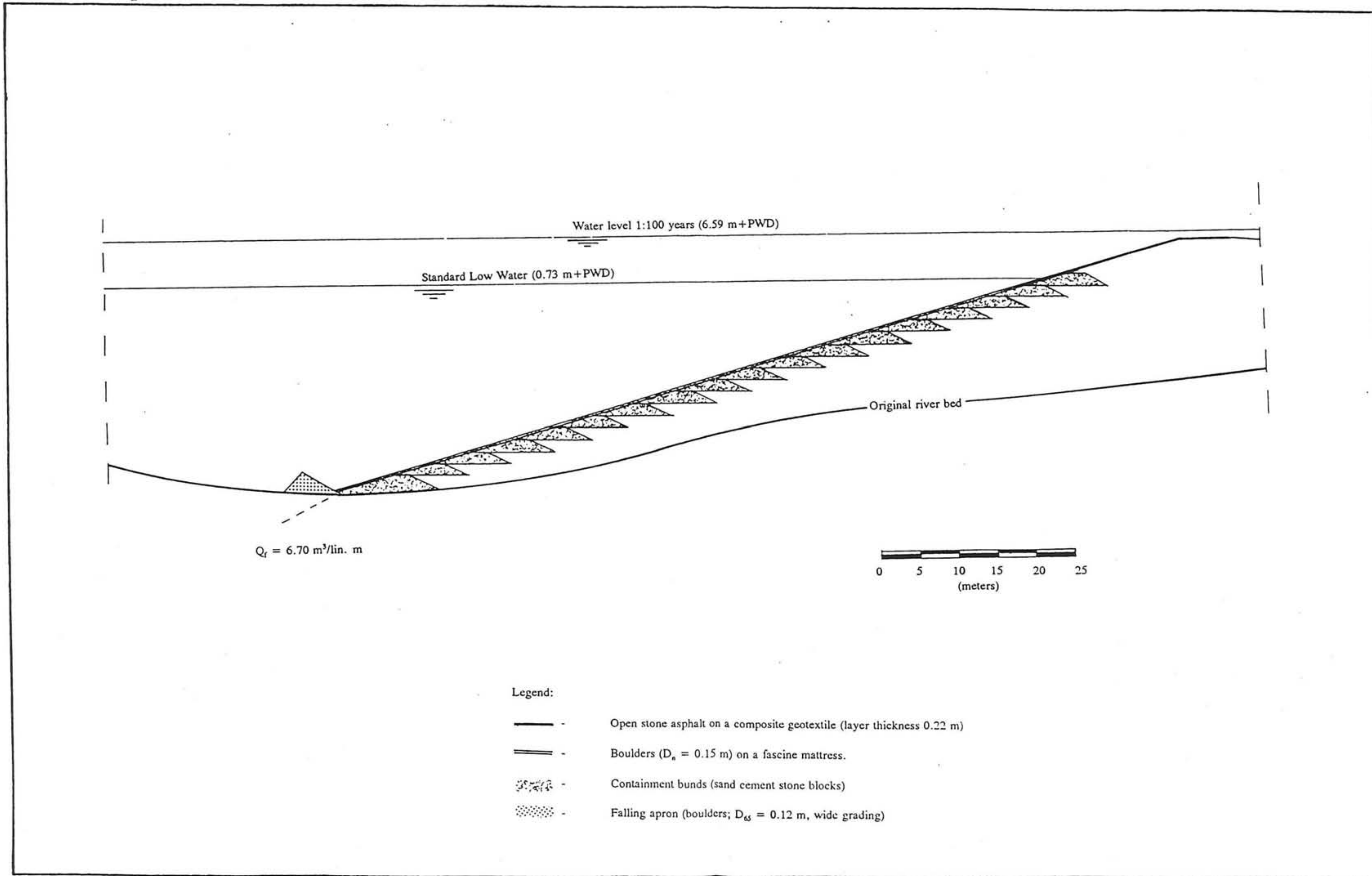


Figure II.15: Alternative I and II, guide bank; cross section.

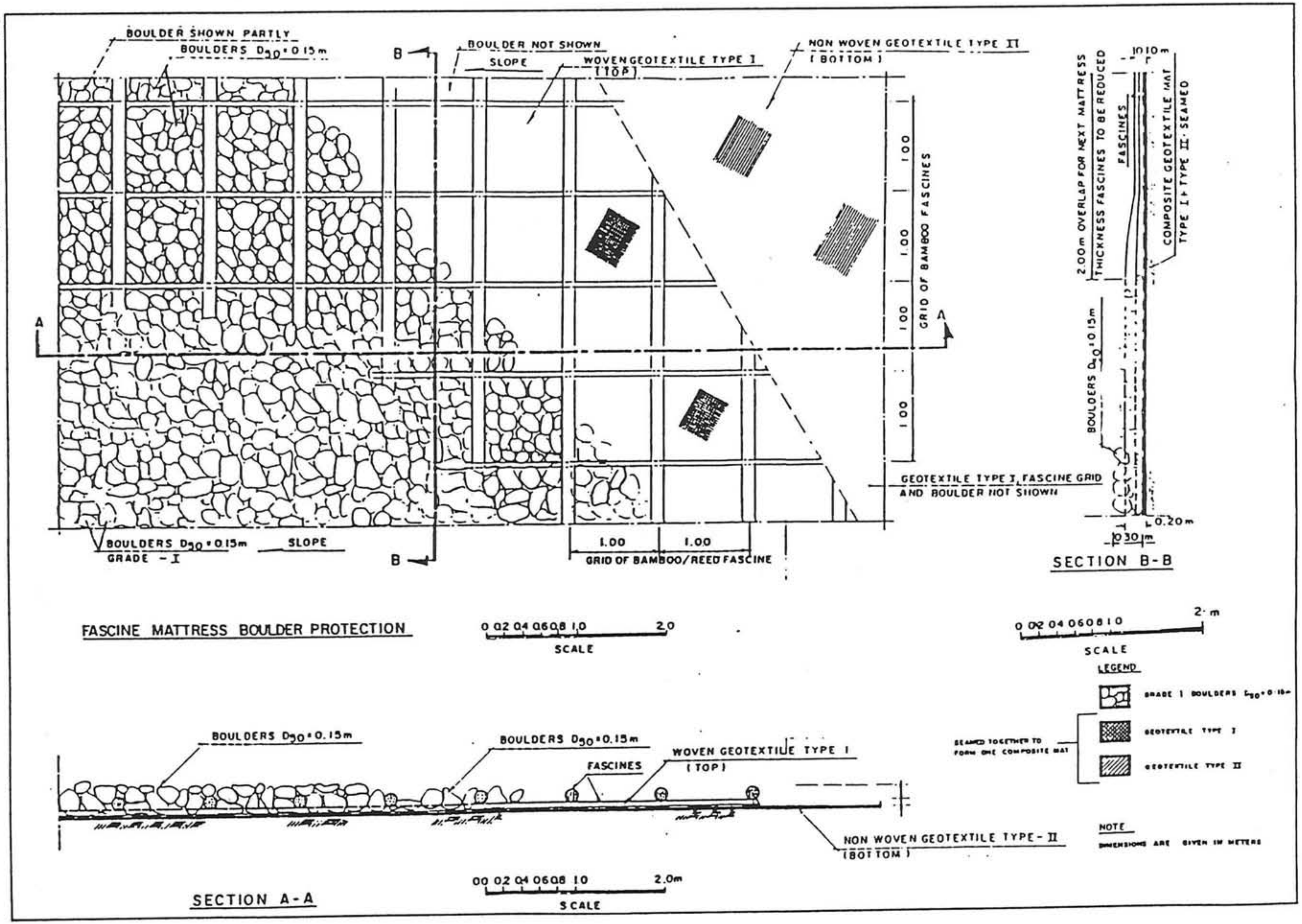


Figure II.16: Typical fascine mattress.

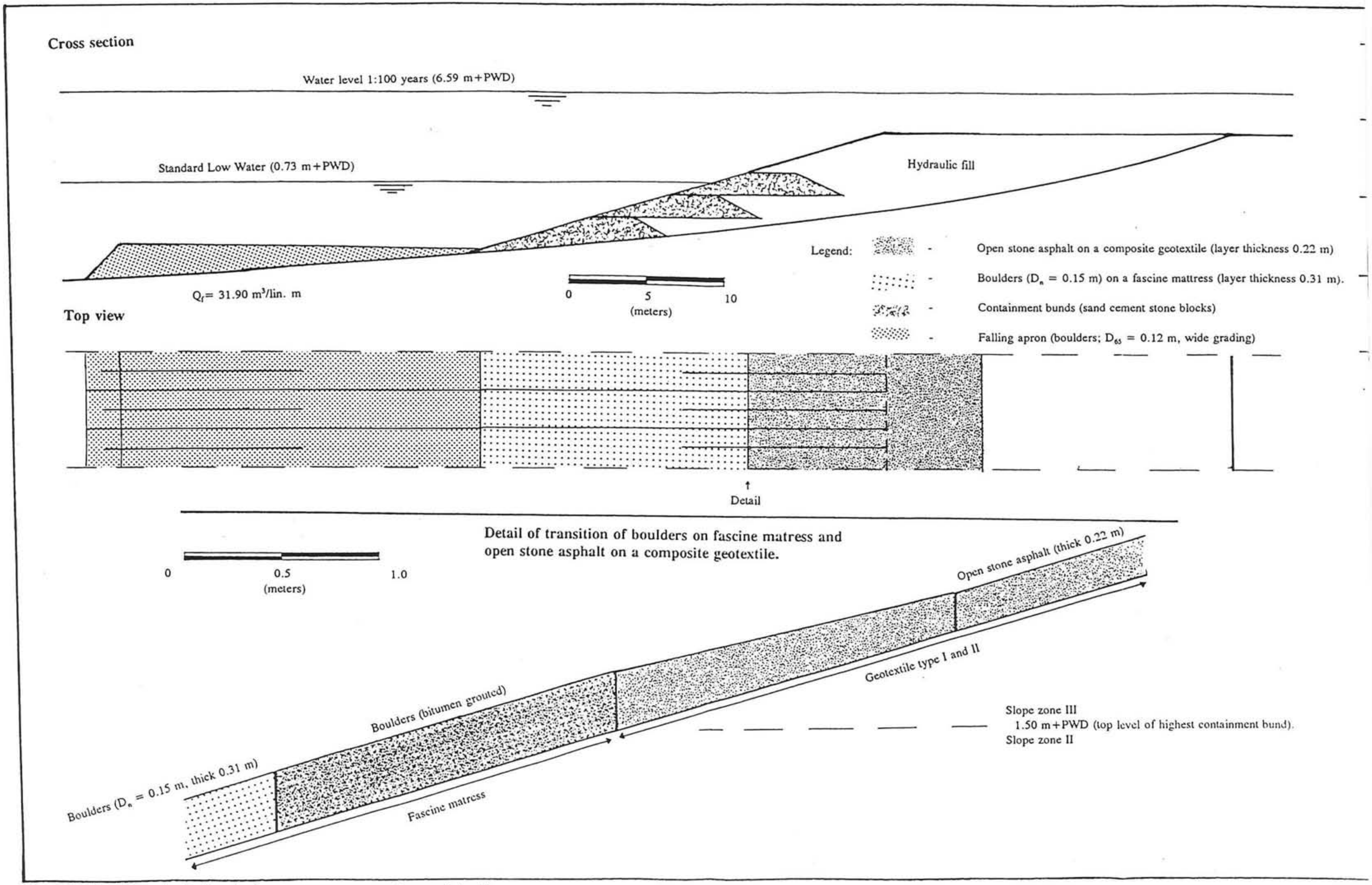


Figure II.17: Alternative 1, river bank revetment; cross sections and detail.

8.5.2 Alternative II

Here the same holds as for alternative I. As part of the structures is constructed from land and in order to facilitate possible maintenance operations here a crest width of 6.0 m for all structures is applied.

- Guide bank (sand core)

Here for the guide bank the same gradings are applied as for the guide bank for alternative I (Section 8.5.1). Also for the extension at the upstream side along the embankment of the approach road to the former Ferry Ghat the same gradings can be applied. The calculations for slope protection materials resulted in a higher requirement for slopes in the flow direction. However, this stricter requirement yields especially for downstream slopes where the flow over the structure can be directed downwards thereby decreasing the stability of the protection material. For this slope being located at the upstream side the same grading can be applied as for the slope parallel to the flow direction.

Some additional remarks have to be made.

- At the upstream end an extension is foreseen to be able to deal with the foreseen erosion. The total length of the landward structure is 200 m. This is equal to the length of the groyne upstream.
- Contrary to the guide bank for Alternative I here at the upstream curve the apron section is dimensioned to a scour level of 30 m-PWD. As the guide bank in this alternative is a detached structure (not connected to any river bank revetment or what so ever) here the extreme scour is foreseen.

The layout of this guide bank is given in Figure II.18. The cross section through the deepest point is similar to the cross section given in Figure II.15.

- Long groynes (sand core)

It is remarked already, when determining the design scour levels, that the first groyne upstream can be expected to be attacked more severely than the other groynes. Therefore an extra safety margin is provided. Also the head of a groyne is attacked more severely than the body. In order to obtain a smooth flow around the head here the slope is reduced to 1:7. A layout and cross sections are given in Figure II.19 and II.20.

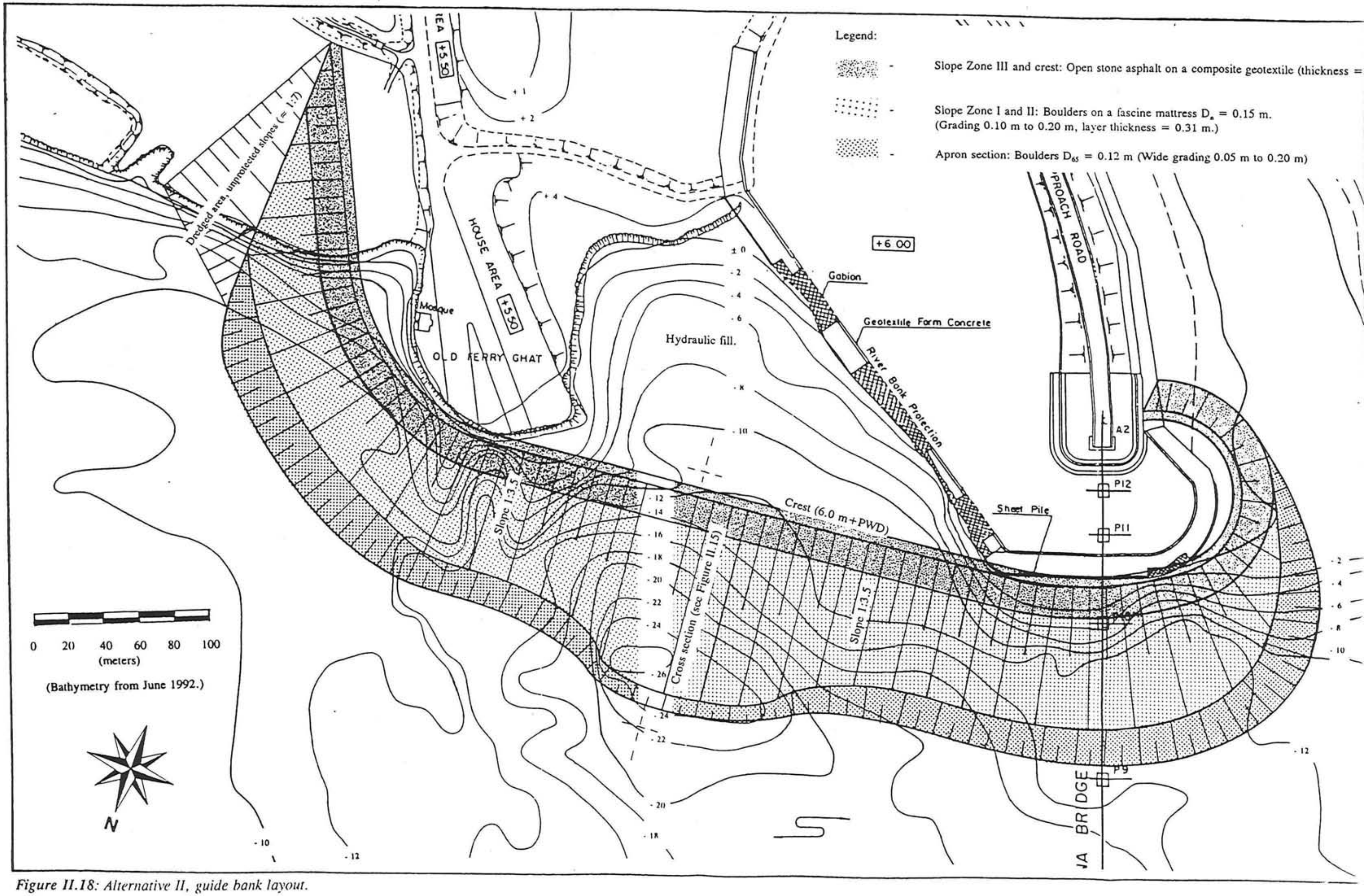


Figure II.18: Alternative II, guide bank layout.

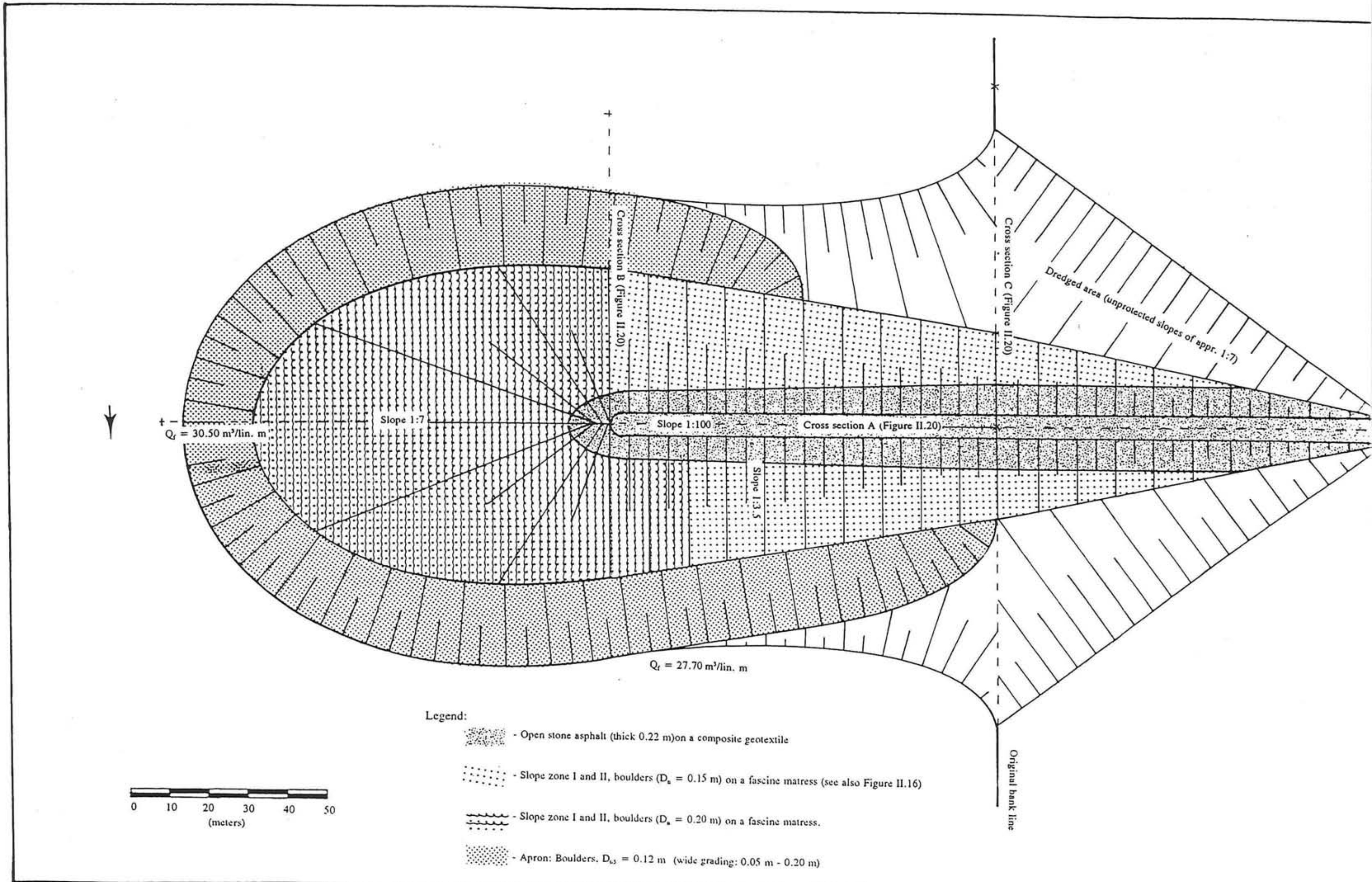
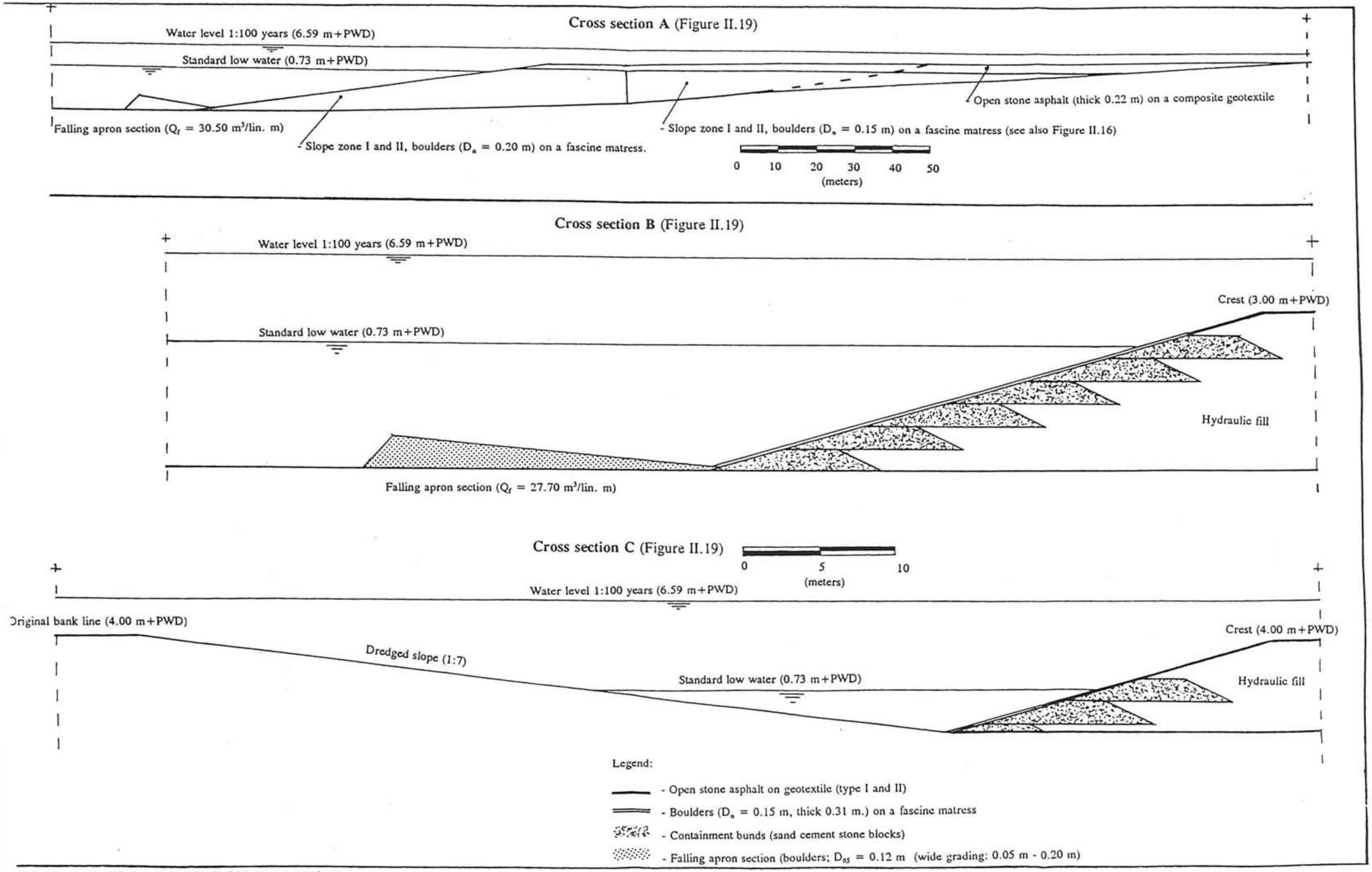


Figure II.19: Alternative II, long groyne (LG2); top view.



20: *Alternative II, long groyne (LG2); cross sections.*

8.5.3 Alternative III

As here the construction of the protection layers can also be done water borne here no crest is applied.

- Guide bank (sand cement stone core)

For the guide bank from the general layout (Figure II.3) the crest line at the river side is taken as the crest line. This enables the construction of the guide bank over the remainders of the present left abutment and the former ferry ghat.
Due to the steeper slope (1:1.5) the toe of this guide bank does not reach the deepest scour hole. Behind the guide bank no outfill is placed and also no slope protection is foreseen as here the hydraulic loads are minimal. Furthermore considerations are similar as for the other guide bank options. The layout and cross section are given in the Figures II.21 and II.22.

- Short groynes (sand cement stone core)

For the considerations see the remarks in Section 8.5.2. Here a front slope at the head of 1:4 is foreseen. The layout and cross sections are given Figure II.23.

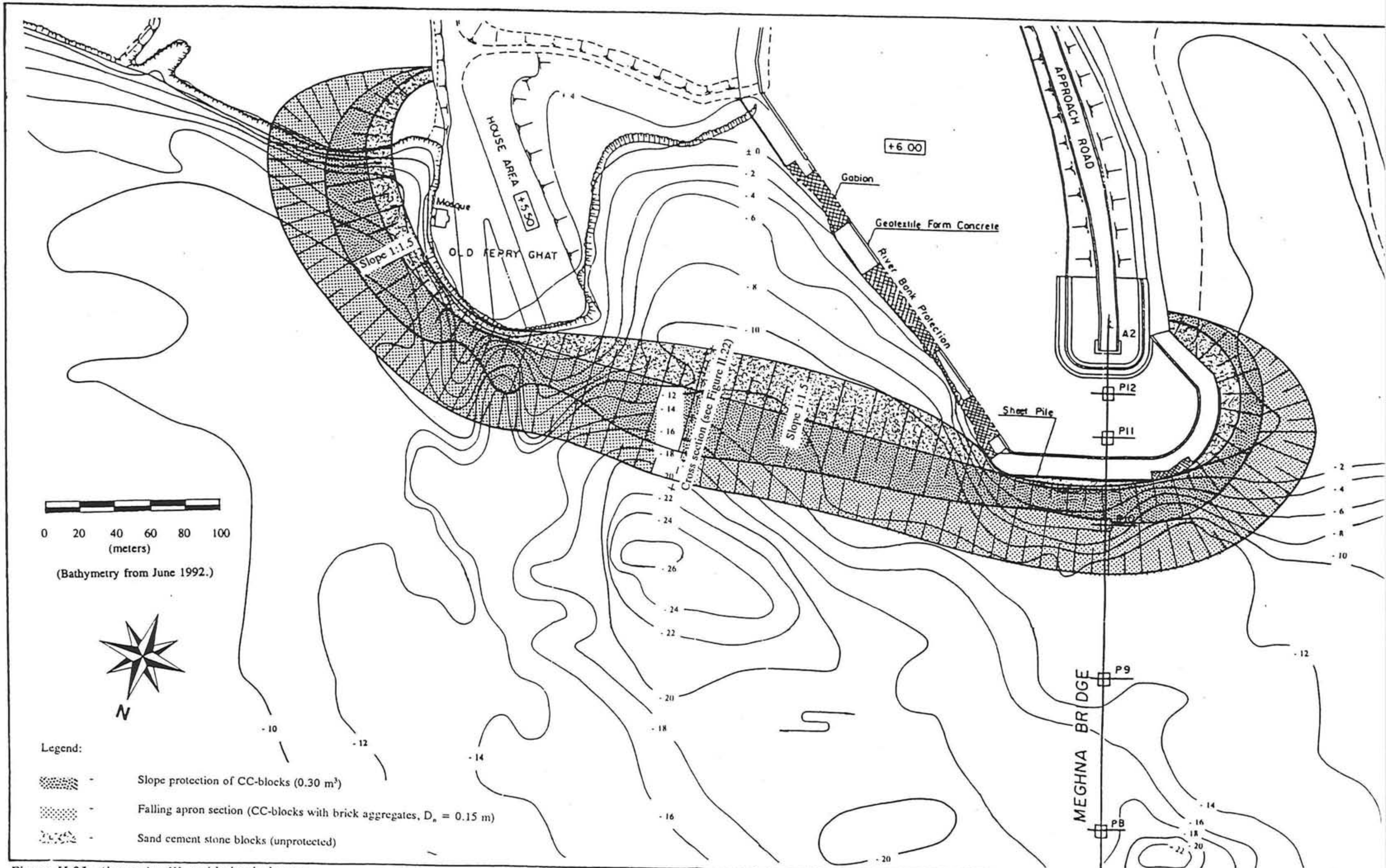


Figure II.21: Alternative III, guide bank; layout.

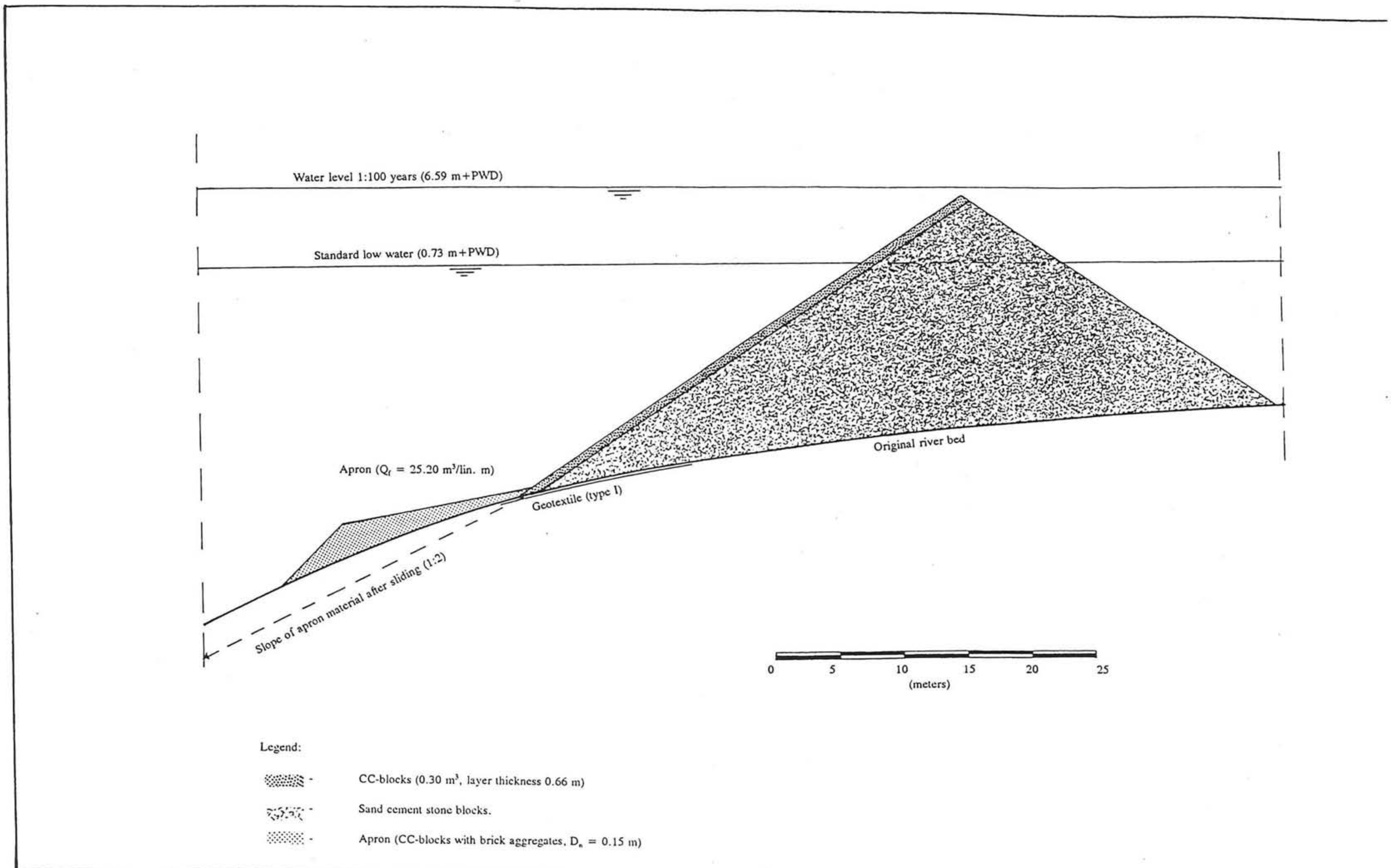
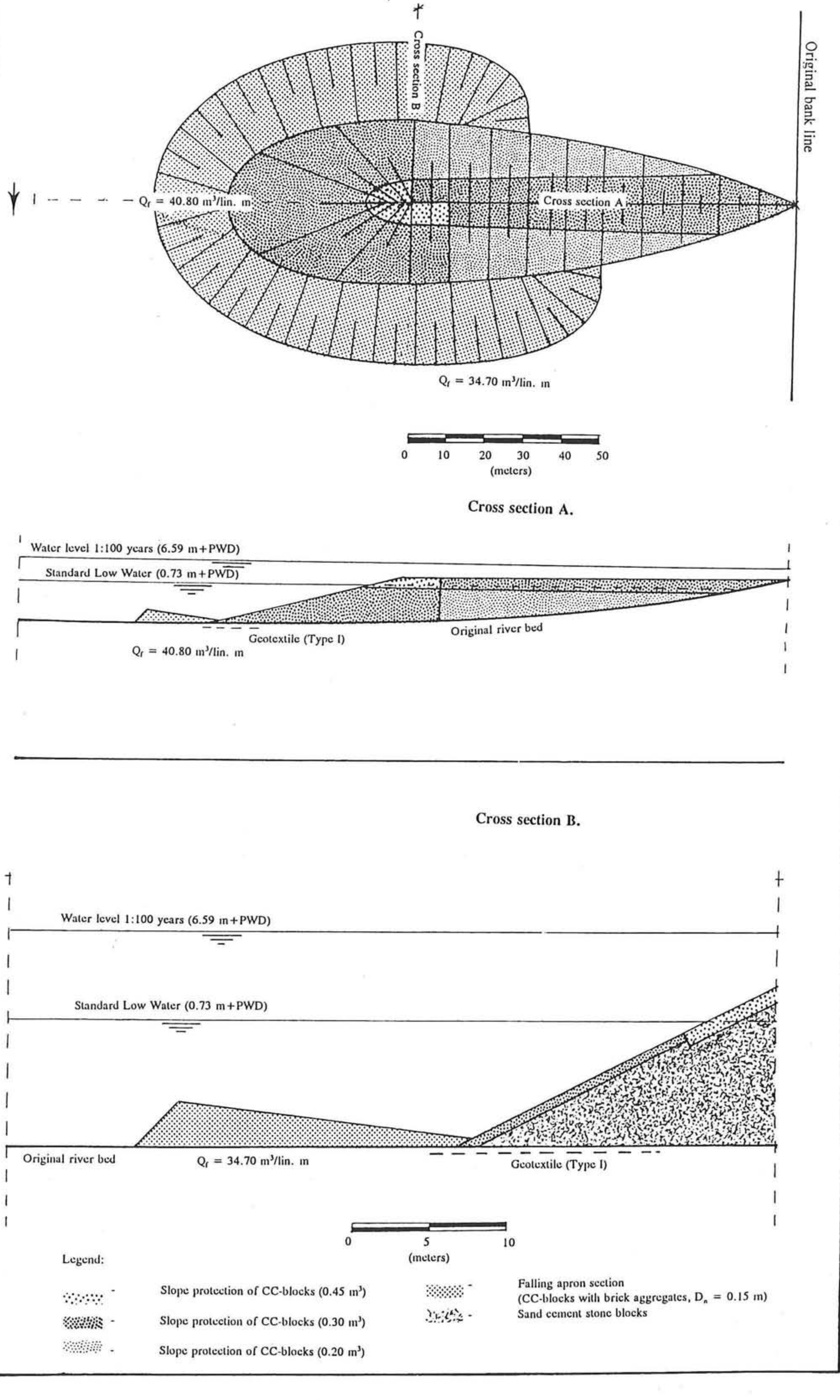


Figure II.22: Alternative III, guide bank; cross section.



I.23: *Alternative III, short groyne (SG4): top view and cross sections.*

8.6 Costs for the alternatives

In order to make a comparison of the costs for the various alternatives preliminary cost estimates are elaborated and presented in Table II.24 to II.26. In these estimates some items are not included as these are similar for all alternatives and therefore not of interest when comparing the alternatives.

Examples of such items are:

- provisional sums,
- clear site and reinstate,
- contractors cost and supervision.

The unit prices, as used for these preliminary cost estimates, are estimates derived from ref. [27]. When in the cost estimates volumes are mentioned these are volumes including the pores. For dumped blocks as sand cement stone- or CC-blocks a porosity of 35% is assumed. This is a conservative estimate.

Table II.24: Preliminary cost estimate for alternative I.

ALTERNATIVE I.				
Item	Quantity	Unit	Unit price (US$)	Cost (US$)
GUIDE BANK:				
- dredging (hydraulic fill)	775000	m3	4.50	$3,487,500
- cont. bunds (slope zone I&II)	80000	m3	27.00	$2,160,000
- earthwork (slope zone III)	60000	m3	2.50	$150,000
- fascine mattress	40000	m2	16.50	$660,000
- open stone asphalt	15500	m2	35.50	$550,250
- boulders in falling apron	20000	m3	35.00	$700,000
- grouting of boulders	700	m	68.00	$47,600
			A) Sub total:	$7,755,350
RIVER BANK REVETMENT:				
- dredging (hydraulic fill)	200000	m3	4.50	$900,000
- cont. bunds (slope zone I&II)	200000	m3	27.00	$5,400,000
- earthwork (slope zone III)	100000	m3	2.50	$250,000
- fascine mattress	100000	m2	16.50	$1,650,000
- open stone asphalt	35000	m2	35.50	$1,242,500
- boulders in falling apron	75000	m3	35.00	$2,625,000
- grouting of boulders	2930	m	68.00	$199,240
			B) Sub total:	$12,266,740
MISCELLANIOUS:				
- working area/materials	1	(-)	$300,000	$300,000
- MOB/DEMOB (land based equipment)	1	(-)	$75,000	$75,000
- MOB/DEMOB (dredging material)	1	(-)	$2,000,000	$2,000,000
			C) Sub total:	$2,375,000
			Sub total (A+B+C):	$22,397,090
OVERHEAD:				
- physical contingencies	17.5	%		$3,919,491
- contractors margins and fees	28	%		$6,271,185
			D) Sub total:	$10,190,676
			Total (A+B+C+D):	$32,587,766

Table II.25: Preliminary cost estimate for alternative II.

ALTERNATIVE II.					
Item	Quantity	Quantity	Unit	Unit price (US$)	Cost (US$)
GUIDE BANK:					
- dredging (hydraulic fill)	800000		m3	4.50	$3,600,000
- cont. bunds (slope zone I&II)	82750		m3	27.00	$2,234,250
- earthwork (slope zone III)	70000		m3	2.50	$175,000
- fascine mattress	42500		m2	16.50	$701,250
- open stone asphalt	18000		m2	35.50	$639,000
- boulders in falling apron	21000		m3	35.00	$735,000
- grouting of boulders	800		m	68.00	$54,400
				A) Sub total:	$8,138,900
(LONG) GROYNES:	LG1 or LG	LG3			(LG1 + LG2 +LG3
- dredging (hydraulic fill)	40000	30000	m3	4.50	$495,000
- cont. bunds (slope zone I&II)	20000	12500	m3	27.00	$1,417,500
- earthwork (slope zone III)	6000	5000	m3	2.50	$42,500
- fascine mattress	7200	4500	m2	17.00	$321,300
- open stone asphalt	4100	3600	m2	35.50	$418,900
- boulders in falling apron	10500	9750	m3	35.00	$1,076,250
- grouting of boulders	350	250	m	68.00	$64,600
				B) Sub total:	$3,836,050
MISCELLANIOUS:					
- working area/materials	1		(-)	$300,000	$300,000
- MOB/DEMOB (land based equipment)	1		(-)	$75,000	$75,000
- MOB/DEMOB (dredging material)	1		(-)	$2,000,000	$2,000,000
				C) Sub total:	$2,375,000
				Sub total (A+B+C):	$14,349,950
OVERHEAD:					
- physical contingencies	17.5		%		$2,511,241
- contractors margins and fees	28		%		$4,017,986
				D) Sub total:	$6,529,227
				Total (A+B+C+D):	$20,879,177

Table II.26: Preliminary cost estimate for alternative III.

ALTERNATIVE III.					
Item	Quantity	Quantity	Unit	Unit price (US$)	Cost (US$)
GUIDE BANK:					
- sand cement stone	165000		m3	27.00	$4,455,000
- CC-blocks	12000		m3	50.00	$600,000
- CC-blocks (brick aggregates)	27500		m3	40.00	$1,100,000
- geotextile type I	750		lin. m	50.00	$37,500
				A) Sub total:	$6,192,500
(SHORT) GROYNES:	SG1 to SG5	SG6			(SG1..SG6)
- sand cement stone	10500	8500	m3	27.00	$1,647,000
- CC-blocks	3450	2750	m3	50.00	$1,000,000
- CC-blocks (brick aggregates)	7050	6050	m3	40.00	$1,652,000
- geotextile type I	300	200	lin. m	50.00	$85,000
				B) Sub total:	$4,384,000
MISCELLANIOUS:					
- working area/materials	1			$750,000	$750,000
- MOB/DEMOB (land based equipment)	1			$500,000	$500,000
- MOB/DEMOB (water based equipment)	1			$500,000	$500,000
				C) Sub total:	$1,750,000
				Sub total (A+B+C):	$12,326,500
OVERHEAD:					
- physical contingencies	17.5		%		$2,157,138
- contractors margins and fees	28		%		$3,451,420
				D) Sub total:	$5,608,558
				Total (A+B+C+D):	$17,935,058

The considerations, used for the cost computations, are the following:

- For sand cement stone, which is also used for the containment bunds in alternative I and II, a cement percentage of 10 is assumed. This is done with respect to the available (small graded) sand mixes in Bangladesh.
- The heights of the containment bunds is taken 1.5 metres, this is rather low and can lead to problems during construction as this requires a good interaction between two very large construction phases (constructing containment bunds and hydraulic filling) this is further discussed in the Evaluation (Chapter 9). However, a low height for the containment bunds leads to a high saving in the material costs.
- For the fascine mattresses for the groynes in alternative II the price is only slightly higher. This is due to the fact that boulders with a larger diameter cost about 15% more only.
- For both the alternatives I and II it is assumed that the required quantity of boulders can be delivered and will not lead to a price increase. However especially for alternative I where the required amount of boulders is about 140.000 m^3, this is doubtfull. For alternative II the required amount of boulders is about 70.000 m^3. In part I of this Report it was remarked already that quantities up to about 100.000 m^3 will not lead to big difficulties.

From the cost estimates the following conclusions can be drawn:

- Due to the very large quantities of material required for containment bunds and slope protection alternative I is rather costly.
- For alternative II, where less material is required for the river bend fixation works, the costs are significantly lower. Still the total costs are higher than those for alternative III as here also dredging material is required.
- For alternative III however, the main part of the construction can be done as a land based operation. Also due to the steeper slopes, less material is required.

9 EVALUATION OF THE FINAL ALTERNATIVES

9.1 Introduction

The three final alternatives, as detailed in the previous Chapter, can be evaluated according to several criteria. The evaluation is discussed in this Chapter.

The evaluation is done in two steps; firstly all non monetary criteria are identified and their weighing factors are determined (Section 9.2). Next the scores for the final alternatives to these criteria are discussed in Section 9.3. Subsequently a final conclusion is drawn taking also into account the costs for the various alternatives.

9.2 Criteria and weighing factors

- Criteria

The (non monetary) criteria to which the final alternatives are evaluated are the following:

1) **Functionality of the solution**; here the expected quality of the finally constructed situation is meant, this criterion can be divided in the bend alignment (support of the main flow) and the vulnerability to scour of the bank protection works.
2) **Construction**; the required construction techniques and problems expected and the compatibility of the construction techniques with the locally available skills and equipment is considered here. This is expressed by using three secondary criteria: duration, availability and quality control.
3) **Maintenance**; this criterion is also best expressed by using three secondary criteria. These are: monitoring, duration and replacement.
4) **Compatibility with future works.** It can be expected that at a certain moment flood embankment works are going to be constructed or the regulated river scheme is to be extended. When designing the bank protection works this is not taken into account as no securities are guaranteed concerning this subject. However, a certain compatibility with the possible construction of possible future works will increase the overall (long term) value of the solution.
5) **Environment**; the environmental aspects of the various alternatives are expressed by; pollution, environmental impact, and the geometry/colour of the structures.
6) **Human factors**; here subjects as vandalism, mishaps and the social impact of the various alternatives is considered.

Flexibility and durability are not taken as a criterion as these are starting points for the design study. Between the various alternatives no real differences concerning these items can be identified and therefore they are not taken as an evaluation criterion.

- Weighing factors

The weighing factors are determined in the next Table:

Table II.27: Determination of the weighing factors.

	Functionality	Construction	Maintenance	Compatibility	Environment	Human factors	Σ	Weighing factor
Functionality	0	2	3	3	3	3	14	23
Construction	2	0	3	3	3	3	14	23
Maintenance	1	1	0	3	2	2	9	15
Compatibility	1	1	1	0	2	2	7	12
Environment	1	1	2	2	0	2	8	13
Human factors	1	1	2	2	2	0	8	13
						TOTAL:	60	99

The scores in this Table signify the following:

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

9.3 Scores for the alternatives

The scores for the final alternatives to the criteria, determined above, are presented in the Tables below:

Table II.28: Score for alternative I.

Primary criteria	Z (%)	Secondary criteria	X (%)	Y	W
Functionality	24	Bend alignment	50	2	21.6
		Vulnerability to scour	50	1	10.8
Construction	23	Duration	40	1	9.2
		Availability	20	1	4.6
		Quality control	40	2	18.4
Maintenance	15	Monitoring	40	2	12.0
		Duration	20	2	6.0
		Replacement	40	2	12.0
Compatibility	12	Extension regulated river scheme	60	2	14.4
		Flood embankments	40	1	4.8
Environment	13	Pollution	40	2	10.4
		Impact	50	3	19.5
		Geometry/colour	10	1	1.3
Human factors	13	Vandalism	10	2	2.6
		Social impact	60	2	15.6
		Mishaps	30	1	3.9
				TOTAL:	167.1

where:

Z = weight of primary criteria in %,
X = weight of secondary criteria in %,
Y = suitability of the alternative in points;
the scores in the evaluation Table signify the following:
Y = 0 satisfies requirements 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,
W = calculated score in points.

Table II.29: Score for alternative II.

Primary criteria	Z (%)	Secondary criteria	X (%)	Y	W
Functionality	24	Bend alignment	50	3	32.4
		Vulnerability to scour	50	2	21.6
Construction	23	Duration	40	2	18.4
		Availability	20	1	4.6
		Quality control	40	2	18.4
Maintenance	15	Monitoring	40	2	12.0
		Duration	20	2	6.0
		Replacement	40	2	12.0
Compatibility	12	Extension regulated river scheme	60	2	14.4
		Flood embankments	40	3	14.4
Environment	13	Pollution	40	2	10.4
		Impact	50	1	6.5
		Geometry/colour	10	1	1.3
Human factors	13	Vandalism	10	2	2.6
		Social impact	60	1	7.8
		Mishaps	30	1	3.9
				TOTAL:	186.7

where:

Z = weight of primary criteria in %,
X = weight of secondary criteria in %,
Y = suitability of the alternative in points;
the scores in the evaluation Table signify the following:
Y = 0 satisfies requirements 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,
W = calculated score in points.

Table II.30: Score for alternative III.

Primary criteria	Z (%)	Secondary criteria	X (%)	Y	W
Functionality	24	Bend alignment	50	3	32.4
		Vulnerability to scour	50	2	21.6
Construction	23	Duration	40	2	18.4
		Availability	20	3	13.8
		Quality control	40	3	27.6
Maintenance	15	Monitoring	40	3	18.0
		Duration	20	3	9.0
		Replacement	40	3	18.0
Compatibility	12	Extension regulated river scheme	60	3	21.6
		Flood embankments	40	2	9.6
Environment	13	Pollution	40	2	10.4
		Impact	50	2	13.0
		Geometry/colour	10	1	1.3
Human factors	13	Vandalism	10	2	2.6
		Social impact	60	2	15.6
		Mishaps	30	2	7.8
				TOTAL:	240.7

where:

Z = weight of primary criteria in %,
X = weight of secondary criteria in %,
Y = suitability of the alternative in points;
the scores in the evaluation Table signify the following:
Y = 0 satisfies requirements 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,
W = calculated score in points.

The scores (Y) given in the Tables are discussed briefly below:

- The **functionality of the solution** is expected to be the best for alternative II and III, as in these alternatives the course of the river is supported at certain points and the Thalweg is kept at a reasonable distance from the (present) bank line. For alternative I the risk exists that in the future the thalweg will shift towards the toe of the river bank revetment (the deepest scour holes can be expected at the toe of the revetment). Therefore the quality of this alternative is very much dependent on proper sliding of the apron material.
- As far as the **construction** is concerned it can be remarked that for alternative I the required techniques and skills are known (for instance from the works constructed as a result of the Short Term Study) but not locally available. Due to the high amount of containment bunds the duration can be expected to become a problem. For alternative III the techniques are not known yet, but as these do not require high tech material it must be possible to develop them using local labour and equipment. For alternative II the required construction techniques are the most complicated. Besides the remarks made for alternative I (see page 123) the duration of the construction is expected to be equal for the various alternatives. Quality control will be the easiest for alternative III, where the slope protection consists of one layer only.
- All **maintenance** aspects can be expected being equal for alternative I and II. For alternative III however, maintenance will be somewhat easier due to the same reason as under "construction"; the slope protection layer consists of one layer only.
- When comparing the **compatibility** with future works for alternative III an extension of the regulated river scheme (for a possible extended outer bend layout, see Figure II.6)is quite easy to conduct. The groynes can be lengthened very easily and new groynes can be constructed one by one without a large mobilisation of construction material.
  The construction of flood embankments will induce an increase of the scour. For alternative I this will require a lot of extra material for toe protection (along the entire bank revetment the apron quantities must increase) while for alternative III this problem occurs only at the groyne heads. For alternative III the least extra material is required as here the number of groyne heads is the lowest.
- Concerning the **environment** the only difference between the alternatives concerns the environmental impact. This impact is the lowest for alternative I, where only the present situation is fixed. For alternative II, where parts of the hinterland are dredged away, the environmental impact can be expected to be the highest while for alternative III the environmental impact is more moderate.
- The only difference concerning the **human factors** between the alternatives is the large social impact of alternative II where parts of the hinterland are "destroyed".

9.4 Conclusion

A summary of the scores, as calculated in Table II.28 to II.30, is given in the Table below. This Table shows clearly that, when taking only non monetary criteria into account, alternative III is the most attractive one. Alternative I and alternative II have almost equal scores.

***Table II.31**: Summary of scores for the alternatives.*

	Alternative		
	I	II	III
Total score	167.2	186.7	240.7

However, when determining the most appropriate alternative also the cost for the alternatives must be taken into account, this will give an idea of the value for money. A summary from the preliminary cost estimates, as elaborated in the previous Chapter, is given in the Table below:

***Table II.32**: Summary of the preliminary cost estimates for the alternatives.*

	Alternative		
	I	II	III
Preliminary cost estimate (US$1)	32,587,766	20,879,177	17,935,058

The "value for money" is expressed by the score of an alternative divided by the costs of this alternative. It must be emphasized that this parameter is only an indicator and has no physical meaning. The resulting values for the alternatives are presented in the next Table.

***Table II.33**: "Value for money" for the alternatives.*

	Alternative		
	I	II	III
Value for money (1/US$1)	5.13	8.94	13.42

From this Table it can be seen that alternative III gives the best "value for money". The margins to the other alternatives are quite high.

As alternative III has the best score for both the costs and the non monetary evaluation it is recommended to implement alternative III in order to solve the problems at the location of the Meghna Bridge.

REFERENCES[9]:

[1] **Meghna River Bank Protection Short Term Study**: Interim Report / Haskoning, Delft Hydraulics, Bets.- March 1990.

[2] **Meghna River Bank Protection Short Term Study**: Final Report / Haskoning, Delft Hydraulics, Bets.- February 1992.

[3] **Consulting engineering services for the Meghna Bridge construction project**: Inception Report / Pacific Consultants International, Nippon Koei Co. Ltd.- July 1985.

[4] **The Meghna Bridge Construction Project**: Volume I, Detailed Design Report / Pacific Consultants International, Nippon Koei Co. Ltd.- February 1986.

[5] **The Meghna Bridge Construction Project**: Volume II, Drawings / Pacific Consultants International, Nippon Koei Co. Ltd.- February 1986.

[6] **The Meghna Bridge Construction Project**: Final Report, Volume 1: Main Report, and Volume II: asbuilt drawings / Pacific Consultants International, Nippon Koei Co. Ltd.- March 1991.

[7] **Basic design study on the project for protecting revetment on the bank of Meghna River**: Interim Report / Pacific Consultants International, Nippon Koei Co. Ltd.- September 1992.

[8] **Flexibele waterbouwkundige constructies**: Lecture notes f4 (in Dutch, previous version) / J. Bouwmeester.- Delft; Delft University of Technology, July 1989.

[9] **Introduction to bed bank shore protection**: Lecture notes f4 (new, actual version) / G.J. Schiereck.- Delft; Delft University of Technology, October 1992.

[10] **Scouring** / H.N.C. Breusers & A.J. Raudkivi.- Rotterdam, Brookfield; A.A. Balkema, 1991.

[11] **Jamuna Bridge Project, phase II study**: river training works, design report - volume II / RPT, NEDECO, BCL.- May 1990.

[12] **Jamuna Bridge Bank Protection Research Project** / CUR.- Gouda, August 1988.

[13] **Principles of River Engineering** / Jansen, P.Ph.(ed).- London; Pitman Publishing Company, 1979.

[14] **Manual on river behaviour, control and training**: Publ no 60. / D.V. Joglekar.- New Delhi, India; POONA Central Board of Irrigation and Power, 1971.

[15] **Experiments on design and behaviour of spur dikes**: Proc. IAHR Conf. Minnesota (p. 145-159) / M. Ahmad.- Minnesota; IAHR, 1953.

[16] **Jamuna Bridge Project, phase II study**: design report, volume II / RPT, NEDECO, BCL.- May 1990.

[9] The sequence of all (available) data is as follows: **Title**: subtitle (comment) / author(s).- place(s); publisher, date.

[17] **Fluvial processes in Geomorphology** / L.B. Leopold, M.G. Wolman and J.P. Miller.- San Fransisco, London; W.H. Freeman and Company, 1964.

[18] **Zandcementsteen in de waterbouw**: demonstratieproject, 2^e druk, herziene versie (in Dutch) / CUR.- Gouda, February 1990.

[19] **Zandcementsteen in de waterbouw**: voordrachten workshop (in Dutch) / CUR.- Gouda, September 1990.

[20] **Grondmechanica** (in Dutch) / A. Verruijt.- Delft; DUM, 1987.

[21] **Rivieren**: Lecture notes f8 (in Dutch) / M. de Vries.- Delft; Delft University of Technology, 1990.

[22] **Handboek voor dimensionering van gezette taludbekledingen**: rapport 155 (in Dutch) / CUR, TAW.- Gouda, 1992.

[23] **Rekenregels voor het waterbouwkundig ontwerpen** (in Dutch) / Rijkswaterstaat, directie S&S, Waterbouw.- Utrecht, May 1990.

[24] **Manual on the use of rock in coastal and shoreline engineering** / CUR, CIRIA.- Gouda, 1991.

[25] **Kursus: "Oever en dijkbekleding"** (in Dutch) / Stichting Postakademisch Onderwijs Civiele Techniek en Bouwtechniek.- Delft, April 1989.

[26] **Stability of breakwater armour layers, deterministic and probabilistic design**: Delft Hydraulics Communication No. 378. / J.W. van der Meer and K.W. Pilarczyk.- Delft, February 1987.

[27] **Meghna River Priority works**: interim update of the preliminary cost estimate - contracts no M-1 & M-2 / Haskoning, Bets.- Nijmegen, March 1993.

[28] **User manual for Biseis v. 1.0**: stability program for circular slip surfaces / Haskoning.- Nijmegen, May 1989.
