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Bangladesh Water Development Board

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River Training Studies of The Brahmaputra River

Draft Final Report

January 1993

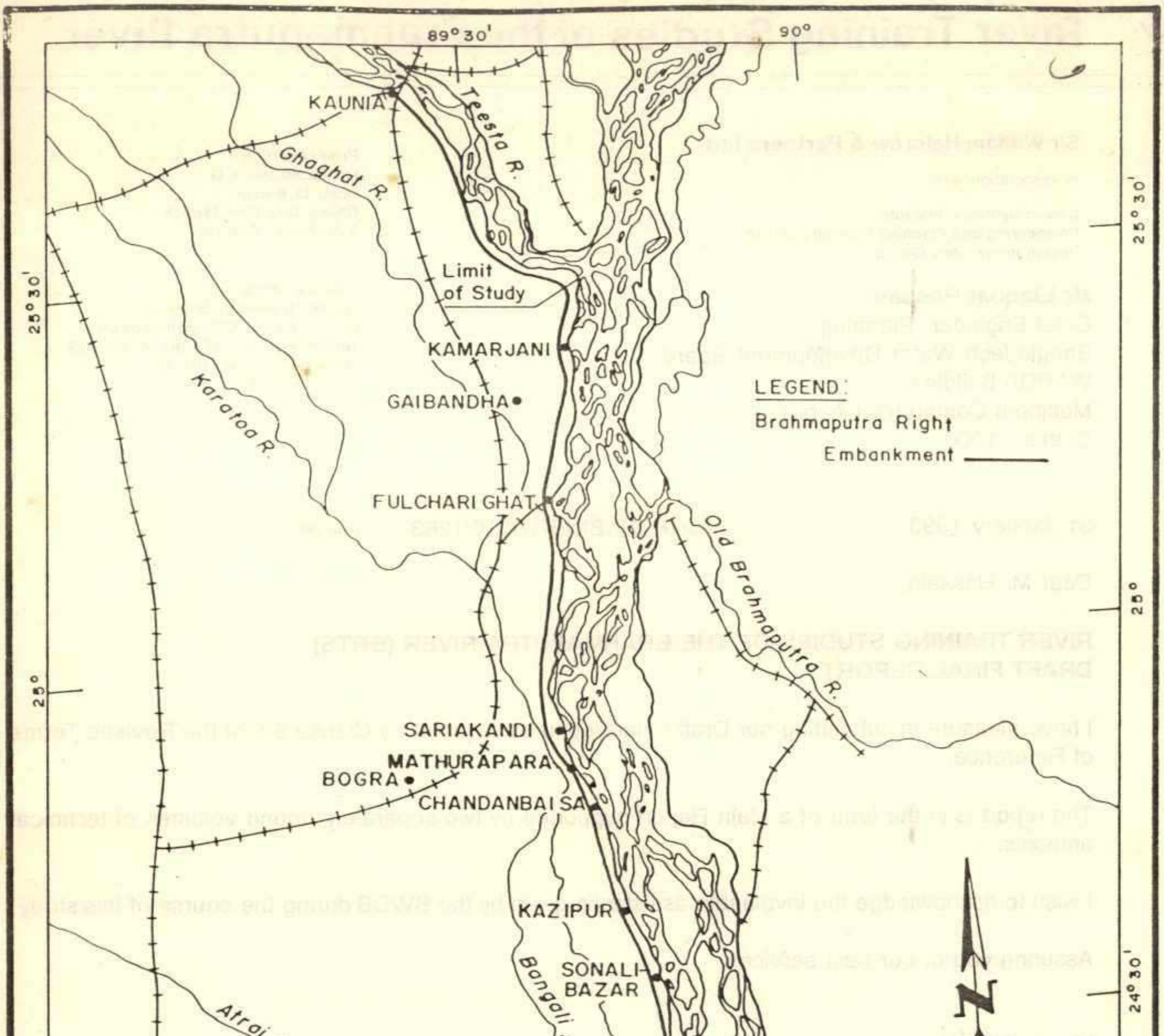
Main Report

Sir William Halcrow & Partners Ltd.
in association with

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

ACKNOWLEDGEMENT

Particular thanks are due to the large number of BWDB staff and others who have contributed to the success of this study. Without their active cooperation the task could not have been accomplished.



Brahmaputra River Study Area

Halcrow/DHI/EPC/DIG *November 1992*

River Training Studies of the Brahmaputra River

Sir William Halcrow & Partners Ltd

in association with

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

Project Office
House 89, Block D
Road 13, Banani,
Dhaka-1213, Bangladesh
Telephone : 608516

Halcrow Office:
House 76, Road 17, Block E
Banani, Dhaka-1213, Bangladesh
Telephone: 606737, 600850, 884629
Telex 642578 HALBD BJ
Tele Fax + 880-2 883663

Mr Liaquat Hossain
Chief Engineer, Planning
Bangladesh Water Development Board
WAPDA Building
Motijheel Commercial Aarea
Dhaka - 1000

31 January 1993 Our ref AP/BD/BRT/320/1263 Your ref

Dear Mr Hossain,

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER (BRTS)
DRAFT FINAL REPORT

I have pleasure in submitting our Draft Final Report as required by Clause 5.6 of the Revised Terms of Reference.

The report is in the form of a Main Report supported by two separately bound volumes of technical annexes.

I wish to acknowledge the invaluable assistance given by the BWDB during the course of this study.

Assuring you of our best services.

Yours sincerely,

M R West
Team Leader

cc Mr M H Siddiqi, Chief Engineer, FPCO
Mr S A M Rafiquzzaman, Director Planning (General), BWDB, Dhaka
Mr S A Rana, Head of Agriculture, World Bank RMB, Dhaka
Mr O J Jensen, Resident Manager, DHI
Mr Zeaul Huq, Managing Director, EPC
Mr Faisal Mahboob, Managing Director, DIG
Prof. M K Alam, Team Leader, BUET Advisory Group
Mr V R Baghirathan, Regional Director, Halcrow, Dhaka
Mr A P G Russell, Chief Engineer Irrigation, Burderop Park, UK

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

DRAFT FINAL REPORT

GENERAL CONTENTS

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

RIVER TRAINING STUDIES OF THE BRAHMAPUTRA RIVER

DRAFT FINAL REPORT: MAIN REPORT

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ABBREVIATIONS

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BIWTA	-	Bangladesh Inland Water Transport Authority
BRE	-	Brahmaputra Right Embankment
BRTS	-	Brahmaputra River Training Study
BWDB	-	Bangladesh Water Development Board
DHI	-	Danish Hydraulic Institute
EIA	-	Environmental Impact Assessment
EIRR	-	Economic Internal Rate of Return
EMP	-	Environmental Management Plan
FAP	-	Flood Action Plan
FIDIC	-	Federation International des Ingenieurs-Conseils
FPCO	-	Flood Plan Coordination Organisation
GOB	-	Government of Bangladesh
ICB	-	International Competitive Bidding
IDA	-	International Development Association (World Bank)
JMB	-	Jamuna Multipurpose Bridge
JMBA	-	Jamuna Multipurpose Bridge Authority
LCB	-	Local Competitive Bidding
LWL	-	Low Water Level
NPV	-	Net Present Value
PIANC	-	Permanent International Association of Navigation Congresses
RE	-	Resident Engineer
RRI	-	River Research Institute
TOR	-	Terms of Reference

SUMMARY

The Draft Final Report has been compiled to explain how the various elements of the Brahmaputra River Training Studies have been carried out, and to record how the conclusions reached have contributed to the formulation of the Master Plan for stabilisation of the right bank of the river and hence protection of the Brahmaputra Right Embankment in the long term.

These studies have further led to the selection of locations for the implementation of river training measures in the short term, and to the design of appropriate structures.

The hydrological studies have helped to define the general hydrological characteristics of the Brahmaputra River System in Bangladesh, and to provide pertinent data for the mathematical and physical modelling, and for the morphological and engineering studies.

The main forms in which mathematical modelling has contributed to the study include:

- the 1-D hydrodynamic modelling programme to determine design water levels, the effects of constriction and confinement of the river, velocity probability distribution, boundary conditions for 2-D modelling, water level and discharge data for the morphological studies and quantification of the effect of breaches in the BRE

- 1-D morphological modelling to assess the magnitude of long term channel geometry changes associated with different levels of river containment

- 2-D modelling to investigate morphological processes such as bend scour, confluence scour, bifurcation, scour around structures and the influence of the Jamuna Bridge embankments. This part of the study has been carried out in close coordination with the physical modelling programme, the two approaches complementing each other.

What is the most appropriate?

The extensive physical modelling programme, undertaken at the River Research Institute, Faridpur, has assisted with the identification of the most appropriate layout for river training works to suit the particular conditions encountered within the study reach of the Brahmaputra River, with the derivation of design values for key hydraulic parameters such as near-bank velocity and scour depth, and with the evaluation of the performance of alternative arrangements for protective layers and falling apron.

Velocity measurements taken under the BRTS river survey programme have provided data for calibration and verification of the 2-D modelling system, and have been used in the calculation of discharge. Velocity distribution is highly variable in the Brahmaputra and a probabilistic approach has been adopted to determine the near bank velocity used in design.

The silt fraction, or wash load, carried in suspension, is not considered important in terms of channel morphology; the sand fraction is transported by a combination of true bed load, dune movement and - evidence suggests primarily - by the movement of massive sandbars.

give in brief this informati

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The morphological studies have covered a wide range of activities providing an essential insight into the long term trends of the river in terms of channel geometry and planform, and into the processes involved in its behaviour, including the development of aggressive bends and the longer term bankline movement. These studies have been planned as one element of a fully integrated study programme including mathematical and physical modelling, and values have been derived for the key parameters required for the design of river bank stabilisation and training works.

Reasoning

It appears that the present westward movement of the right bank will continue for several decades, probably at a similar rate to that experienced over the last 35 years; at the sametime the river is becoming steadily wider. Guidelines have been established for the prediction of the evolution of aggressive bends and a direct link has been demonstrated between bank erosion and char building, although there is often a time lag between the two processes.

This understanding of the bank erosion process, the role of bends and the dominant waveforms with which they are associated, provides the essential background for the planning of stabilisation works at the study reach level and the setting of priorities for early implementation of individual elements.

The Draft Final Report contains a summary of the Master Plan, including the long term and short term measures for river training, implementation measures, the basis for cost analysis, operation and maintenance and finance arrangements. The Master Plan summary also includes a description of the situation as it exists in terms of river behaviour, with predictions for river's future behaviour and an assessment of the consequences of different levels of intervention; further consideration is given to the status of the BRE, to sociological, economic and environmental aspects of implementation, river transport, and construction management. Brief recommendations for follow-up action are included.

The principal design considerations are summarised, with details of how the various parts of the Study have been drawn upon to provide design parameters and guidance as to structural performance. A description of the proposed river training structures is included, together with a description of the methods which may be employed in their construction.

1. INTRODUCTION

1.1 Background

The security of the Brahmaputra Right Embankment (BRE) and consequently the area protected by the BRE has been seriously threatened by continued bank erosion. Since the economic and social consequences of the present approach in dealing with the problem may not be acceptable in the long-term, the Government of Bangladesh (GOB) has commissioned the River Training Studies of the Brahmaputra River (BRTS) to seek a long-term strategy for the protection of the BRE. The project, funded under the IDA sponsored Bangladesh Second Small Scale Flood Control, Drainage and Irrigation Project (Credit No. 1870 BD), is being executed by the Bangladesh Water Development Board (BWDB).

BWDB appointed Sir William Halcrow & Partners Ltd. (Halcrow) in association with Danish Hydraulic Institute (DHI), Engineering and Planning Consultants Ltd. (EPC) and Design Innovations Group (DIG) to undertake this three-year study.

An advisory group from the Bangladesh University of Engineering and Technology (BUET) has been working with the Consultant's Team. The River Research Institute (RRI) were nominated to carry out the physical modelling studies required by the BRTS.

A Letter of Intent was issued by the BWDB on 24th January 1990 to commence the project. The contract for consultancy services was signed between BWDB and Halcrow on 12th March 1990. The Consultant commenced the project on 6th February 1990 by making arrangements to mobilize staff and establish an office and support facilities. Staff inputs commenced on 1st March 1990.

In November 1989, a five year Flood Action Plan (FAP), coordinated by the World Bank, was initiated with the Government of Bangladesh. The FAP is connected with an initial phase of studies directed towards the development of a comprehensive system of long-term flood control and drainage works. Priority has been given to the alleviation of flooding from major rivers, of which the Brahmaputra is a significant source. The BRTS forms component No 1 of a total of 26 components comprising the FAP during the plan period 1990-1995.

Prior to commencement of the consultancy services, the Consultant was instructed by the BWDB to review and revise its work programme and staffing schedule during the inception phase to integrate the project within the FAP.

In accordance with the Terms of Reference, the Inception Report was submitted three months after the commencement of staff inputs, at the beginning of June 1991.

The report included a comprehensive review of the original programme of work in the light of the Flood Action Plan, the delay in project commencement and a greater appreciation gained during the inception phase. This was accepted by the World Bank, BWDB and other related organizations of GOB at a meeting held on 28th July 1990.

A revised staffing schedule was agreed with BWDB at a meeting with Chief Engineer Planning and others on 13th August 1990. The Revised Terms of Reference (TOR) were approved by FPCO on 6th October; the First Amendment to the consultancy contract was finalised on 29th October.

1.2 The Project

The Brahmaputra-Jamuna river system is the largest and most important in Bangladesh, accounting for more than 50 percent of the total inflow into the country from all cross border rivers. Its dry season planform (March 1992) is shown in Plate 1.

The Brahmaputra moved to its present course about 200 years ago. It is a braided river without fixed banks and with frequently shifting channels. Short-term channel migration can be quite drastic with annual rates of movement as high as 800 m. The bank erosion process is a complex mechanism and is influenced by a number of factors. In Bangladesh the overall river width varies between 4 km and 15 km. The river cross-section has a highly irregular bed elevation and, within the study reach, the main channel may be up to 35 m deep although the mean value is only 7 to 8 m.

A 220 km long earth embankment, known as the Brahmaputra Right Embankment (BRE), has been constructed on the western bank of the Jamuna River to protect the lands against the ravages of yearly flood. However, every year this embankment has to be retired landward at several places due to bank erosion; a total length of about 140 km of retired embankment has been constructed over the past 20 years.

River erosion is also causing serious problems at specific locations such as ferry crossings, where the terminal stations (ghats) have to be shifted as a result of eroding river banks.

The principal objective of the Study is to formulate a master plan for the long-term protection of the Brahmaputra Right Embankment (BRE) but also included in the assignment is the design of short-term measures at critical sections along the right bank for early implementation.

The Master Plan is required to address possible alternative measures to river training works - such as embankment retirement or combinations of both types of measures - for ensuring the BRE's satisfactory performance. The selection of the recommended alternatives are to be based on technical and economic analyses, and social and environmental considerations.

1.3 Other BRTS Reports and Technical Notes

Following publication of the Inception Report in June 1990, a Technical Note entitled "Working Paper on 2-D Modelling" was prepared and issued in December 1990. This set out in greater detail the way in which 2-D modelling was to be utilised and gave full details of the modelling system and how it was being set up and calibrated using the river survey data collected for this purpose. The paper also included full details of the associated 1-D modelling system.

A Discussion Paper entitled "Selection of Locations for Priority Works" was issued in December 1990. This provided the background to the problems of bank erosion as perceived at that time and set down the initial screening criteria that were being used to select locations along the river bank for priority attention.

The BRTS First Interim Report (April 1991) presented the findings from the first river survey campaign, carried out during the 1990 monsoon season and the following low flow period, and described the first insights into the behaviour of the river derived from this. The proposed scope and form of the master plan was set out and subsequently ratified by the BWDB, FPCO and World Bank.

Draft detailed designs for priority bank stabilisation works at six locations were prepared together with draft tender documents suitable for international competitive bidding and submitted for discussion and comment in October 1991.

The Second Interim report, issued at the end of the 22nd month of the 36 month study duration (December 1991), marked the substantial completion of all field work although some further near-bank river surveys were carried out during the early months of 1992. By this time the mathematical modelling components were approaching completion and the physical modelling was well advanced; the interim results from both these components were presented in Annexes to the report. Together with the outcome of the morphological studies, these results led to a very much better understanding of the river's characteristics and probable future behaviour. With the combination of this and an improved appreciation of the sociological problems and the agro-economic impacts of breaches in the BRE, it was possible to carry out a more detailed economic assessment of the six priority locations identified earlier. The results of this analysis were presented in Annex 6 to the report and formed the basis for the subsequent preparation of the World Bank financed River Bank Protection Project.

In accordance with the TOR requirement, the formal completion report on the river surveys formed Annex 1 to the Second Interim Report. User Guides for the 1-D modelling, 2-D modelling and the River Survey Databases were compiled and formed the basis for training of BWDB and SWMC staff and a formal handover took place in April 1992, marking the actual completion of these components.

A revised Design and Construction Management Report was prepared and issued in August 1992. This comprised a complete update of the design procedures and criteria in the light of the improved understanding of the river conditions and the results of the physical modelling and also described in detail the construction methodology and scheduling on which the design of the Phase 1A (the Sirajganj and Sariakandi/Mathurapara Components) were based.

Following revision and redrafting of the detailed designs and contract documents for the Phase 1A works, the Environmental Impact Assessment for the priority bank stabilisation works was completed and the report submitted in July 1992. Environmental management measures recommended in the EIA were incorporated in the ICB tender documents.

A further report on Provisions for Operation and Maintenance of the Priority Works was submitted in July 1992.

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The test bed work at RRI under the BRTS physical modelling programme was completed in May 1992 and a series of reports are in the final stages of preparation. These will be summarised to form the physical modelling section of the final Report on Model Studies, a requirement of the TOR, which is due for submission at the end of January 1993. The mathematical modelling section will cover both 1-D and 2-D modelling and present the results of the applications carried out following the issue of the Second Interim Report. A full list of reports and technical notes is attached in Appendix A.

The principal activities since April 1992 have centred on the tasks noted above, the preparation of the Prequalification Document for the Priority (Phase 1A) Works and completion of the morphological studies. The latter has been undertaken in coordination with the work carried out by ISPAN under FAP-19.

1.4 Relationship with other FAP Projects

The other FAP studies that have specific links with the BRTS are:

FAP-2:	North West Regional Study
FAP-3:	North Central Regional Study
FAP-3.1:	Jamalpur Pri[illegible] Project
FAP-9B:	Meghna Left [illegible]otection Project
FAP-12:	Flood Control, Drainage and Irrigation Projects Agricultural Review
FAP-13:	Operation [illegible] Maintenance Study
FAP-14:	Flood Respo[illegible]udy
FAP-15:	Land Acquisition and Resettlement Project
FAP-16:	Environmental St[illegible]
FAP-17:	Fisheries Study and Pilot Project
FAP-18:	Topographic Mapping
FAP-19:	Geographic Information System
FAP-20:	Compartmentalisation Pilot Project
FAP-21:	Bank Protection Pilot Project
FAP-22:	River Training/and Active Flood Plain Management Pilot Project
FAP-24:	River Survey Programme
FAP-25:	Flood Modelling/Management Project

Close coordination has been maintained with all these studies but particular mention should be made of the close working relations that have been sustained with the two neighbouring regional studies (FAP-2 and FAP-3), the very valuable complementary work being undertaken by FAP-19 and the significant contribution made by FAP-25.

While the delayed start of some supporting studies did inevitably have some impact during the course of the Study, this has not been allowed to materially affect progress and results, conclusions and recommendations have been absorbed as they became available.

1.5 Structure of this Report

The contents of this Draft Final Report have been structured to conform with the requirements of Article 5.6 of the Terms of Reference which stipulates that the report shall include:

- a description of the predicted future behaviour of the Brahmaputra River based on results of hydrological, hydraulic and morphological studies, mathematical model studies, physical model studies and master plan studies;

- how the mathematical and physical models helped in preparing the master plan;

- a brief summary of the master plan along with short term and long term river training measures;

- a brief summary of designs of the proposed river training structures.

The outcome of the hydrological, hydraulic and morphological studies are described in Chapters 2, 5 and 6 while the objectives and results of the mathematical and physical modelling and how they relate to the Master Plan are covered in Chapters 3 and 4. These lead on in Chapter 7 to a description of the characteristics of the river and how they may be used predict its future behaviour. A brief summary of the Master Plan is presented in Chapter 8 and Chapter 9 contains an outline description of the design of the proposed river training structures.
This main report is supported by the following technical annexes:

Annex 1 - Sediment Transport and Channel Characteristics
Annex 2 - Morphological Studies

As required by the Terms of Reference a separate Master Plan Report will be issued and will provide further details of the Master Plan and a full description of the designs and tender documents relating to river training works for Phase 1 of the Master Plan.

1.6 Nomenclature

A list of abbreviations and acronyms used is given at the beginning of this report.

The name frequently used for the Brahmaputra River in Bangladesh is the Jamuna. Although the name Brahmaputra is more generally used in this report, the name Jamuna, when it appears, is not intended to imply any differentiation.

2. HYDROLOGICAL CHARACTERISTICS

2.1 Introduction

The objectives of the hydrological studies carried out during the first phase of the BRTS were firstly to define the general hydrological characteristics of the Brahmaputra River system in Bangladesh and secondly to provide pertinent hydrological characteristics for the hydrodynamic and physical modelling, and for the geomorphological and engineering studies.

Specific requirements were the definition of the following relating to the Brahmaputra within the study area:

- general rainfall characteristics on the catchment area in Bangladesh
- general and seasonal water level characteristics
- general and seasonal streamflow characteristics
- seasonal sediment transport characteristics

A number of earlier studies were reviewed and their results provided a valuable background for the present study. A brief description of hydrological activities relating to the major studies is given in Appendix B. The majority of the detailed information was however derived from fresh analysis of historic hydrological data collected by BWDB.

Basic analysis of rainfall, water level, discharge and sediment transport were undertaken to define the seasonal hydrological characteristics of the river. A finer definition of the seasonal variation of hydrological parameters was supplemented by 10-day analysis wherever considered necessary. The long-term fluctuations of the Brahmaputra water regime were described by using monthly quantiles.

The long-term trend in the annual peak flood characteristics (ie. water level and discharge) was investigated by using moving average and cumulative average techniques.

At-site flood frequency analysis was carried out by using the screened annual maximum flood series. A Gumbel Extreme Value Type I (EV1) distribution was fitted to the annual maximum series and the parameters of the distribution, in the first instance, were estimated by the methods of ordinary moments and maximum likelihood. The goodness-of-fit of the distribution was verified using the Kolmogorov-Smirnov test and by graphical plotting. If the moment estimate provided better fit than the likelihood estimate then parameters of the EV1 distribution were estimated by the method of probability weighted moments. However, if the plot revealed the inadequacy of the EV1 distribution in representing the sample data only then the three-parameter General Extreme Value (GEV) distribution was fitted to the at-site data and parameters were estimated by the method of probability weighted moments.

2.2 Some Catchment Characteristics

The Brahmaputra river rises in Tibet on the north slope of Himalayas, flows eastward for about 1,100 km, turns south into the Indian province of Assam, and then turns sharply west for

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about 640 km to the border of Bangladesh (see Figure 2.1). At the border, the river curves to the south and continues on this course to its confluence with the Ganges about 240 km down-stream. The catchment area is approximately 520,000 km^2.

Snowmelt accounts for most of the flow of the Brahmaputra, but rainfall in Assam and in the northern part of Bangladesh contribute significantly. Consequently the river flow has high seasonal variability.

Three major tributaries, the Teesta, Dudhkumar and Dharla, rise in the Himalayas and flow south-east across the North Bengal plains before entering Bangladesh in its north-west corner. These rivers bring down from the Himalayas large quantities of sediment. They drain a high rainfall area and are subject to flash floods. Some of their principal characteristics are summarised in Table 2.1.

Table 2.1 Characteristics of Major Tributaries

River	Gauging Station	Catchment Area (km2)	Average Daily flow (m3/s)	Annual Maximum Daily Flow (m3/s)
Teesta	Dalia	10,100	850	9,200
Dudhkumar	Pateswari	7,030	500	8,800
Dharla	Taluksimulabari	5,220	500	7,800

The most important distributary of the Brahmaputra is the Old Brahmaputra, the former course of the Brahmaputra over 200 years ago. Farther downstream, just below Sirajganj, another major distributary, the Dhaleswari, leaves the left bank of the Brahmaputra.

2.3 Data Sources

A comprehensive network of hydrological stations covering the Northwest and North Central Regions of Bangladesh has been established by the BWDB and this was the main source of hydrological data for the BRTS analysis. The Directorate of Surface Water Hydrology-I is responsible for hydrological field work whilst the Directorate of Surface Water Hydrology-II is responsible for data storage, processing and disbursement.

Rainfall

A relatively large number of daily recording rainfall stations are available, the majority of which were established during the early 1960's, and these provide a satisfactory data base for the definition of the rainfall characteristics. Since the objective of the rainfall studies is only to provide a general definition of the rainfall characteristic of these areas related to the Brahmaputra river, data from five stations (see Figure 2.2) were collected for detailed analysis. These stations have been selected on the basis of having relatively long periods of continuous

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data of satisfactory quality, and providing wide areal coverage, particularly of the Brahmaputra tributary catchments in the northwest region. Full details may be found in the BRTS First Interim Report.

Water Level Data

An extensive network of water level gauging stations on the Brahmaputra and its tributaries and distributaries has been established by the BWDB. Whilst many of these stations have relatively long periods of more or less continuous data, a number have short periods and/or discontinuous data.

A total of 35 stations were selected for defining the water level characteristics of the Brahmaputra and its major tributaries and distributaries. Of these, 19 are on the Brahmaputra from Noonkhawa (No. 45) to Teota (No. 50.6). Porabari (No. 50) and Sirajganj (No. 49) have the longest records of 36 years each. The data at Bahadurabad Transit spans 32 years.

The remaining 16 stations provide satisfactory definition of the major tributaries and distributaries. In addition water level data for three stations of the Gorai, Arial Khan and lower Meghna representing external boundaries of the BRTS 1-D model were also collected. A summary of the station network for which data were collected is given in Table 2.2. Further details may be found in the BRTS First Interim Report.

Discharge Data

Bahadurabad Transit is the only station where the Brahmaputra discharge is currently being measured and flow data are available since 1956. In the past, the Brahmaputra discharges were measured at Chilmari, Sirajganj and Nagarbari and were continued for varying periods ranging from 1-3 years. These data however would not provide a reliable definition of the river flow characteristics due to their relatively short period. Therefore the available discharge data at Bahadurabad Transit is considered the most suitable for discharge analysis.

The river stage at Bahadurabad Transit is observed five times a day on a regular basis. Discharge measurements are also carried out routinely, usually with a frequency of 2-3 weeks in order to determine and update the rating curve at this location. These measurements are carried out by the traditional current meter approach (2-point method) using a boat. The alignments of the transit lines are determined using sextants. Available rating curves at Bahadurabad Transit were collected to analyse the measured flow characteristics and to determine the year to year variation in rating curves due to changes in the river hydraulics.

Discharge data are also available for major tributaries and distributaries of the Brahmaputra. Data are usually available from the early 1960's, but with discontinuities at some stations. Available daily discharge data were collected for the following rivers: Bangshi, Baral, Karatoa, Dhaleswari, Charlkata, Dudhkumar, Ghagot, Old Brahmaputra and Teesta

Daily discharge data of the Ganges and Meghna were also collected to represent the external discharge boundaries of the 1-D model.

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Table 2.2 Summary of Water Level Data Collection Stations

River System	Data Collected From (Nos. of stations)
Brahmaputra	19
Bangshi	1
Baral	1
Deonai-Charalkata Jamuneswari-Karatoa	3
Dhaleswari	2
Dharla	1
Dudhkumar	1
Fakirni Barnai	1
Ghagot	1
Jamuna	2
Karatoa-Atrai-Gur-Gumti-Hurasagar	2
Old Brahmaputra	1
Teesta	1
Gorai	1
Arial Khan	1
Lower Meghna	1
Total	38

Sediment Data

Sediment data of the Brahmaputra at Bahadurabad Transit was first collected by BWDB in 1956 but sediment investigations on a more rational footing commenced in 1965, using improved techniques and limiting attention mostly to the suspended sediment. The components of the suspended load observed by BWDB fall into two categories. These are:

(a) Suspended bed load (sand fraction) of particle size diameter 0.4 mm to 0.05 mm and

(b) Wash load (clay or silt fraction) of particle diameter less than 0.05 mm.

Available suspended sediment data of the Brahmaputra at Bahadurabad Transit are available for the period 1968-88. These data were observed intermittently on a weekly or fortnightly basis.

2.4 Data Quality Control

The quality control of hydrological data is recognised as an essential step in data analysis. A variety of data error sources exist in the normal process of collecting and archiving hydrological data. The following conventional procedures were applied for checking of data:

(a) Plotting the original data serially by date. This plot can detect the misplaced decimal and help screen out any unrealistic peaks and troughs in hydrographs.

(b) Plotting the data of one station versus the data of a nearby station. This plot may indicate the strength of the relationship between the two stations and may indicate data error which has resulted in changes in that relationship.

(c) Double mass curve plotting. This technique is perhaps the most useful in detecting systematic errors in the data. A change in the relationship may be reflected in a change in the slope of the resulting curve.

Data infilling and extension procedures were used where necessary.

Errors from a variety of sources can be introduced into water level data. For example, in a fluvial river such as the Brahmaputra, it is sometimes necessary to shift the gauge along and across the river course to collect data at all river stages. During such shifting process if the zero datum of the gauge is not properly connected with the nearest available bench mark, the gauged data become susceptible to errors. This type of error tends to occur while shifting the gauge at the rising and falling stage of the river. Such errors could be easily identified if the history of the shifting of the gauge were available. The alternative way of checking the quality of the data is to plot the water level time series of the gauging station along with the time series of its adjacent gauging station. Any significant departure from the general shape of the nearby hydrographs will reveal the error in the data of the gauging site in question.

Another major source of error in the water level data is when the zero datum of the gauge is connected with an unreliable bench mark.

In this study, quality of the 10-day water level data, mean daily water level data and 1-day annual maximum water level data for selected stations were assessed by the following techniques:

(a) 10-day water level time series of the major gauging stations (ie. stations with at least 15 years of data) along the Brahmaputra were plotted to detect any major inconsistency in the data. In general, the data is consistent and of acceptable quality. However, inconsistency in the data in different years at different stations was identified. eg. the 10-day water level data of Kamarjani for the period 1974-76 is not consistent with Noonkhawa and Chilmari.

(b) The mean daily water level data of the Brahmaputra at Bahadurabad Transit were visually inspected with a computer based objective method of ascertaining the most likely errors in the gauged data. The mean daily hydrograph was also plotted serially by date. In addition to detecting obvious data errors, this plot was helpful in screening out the inconsistent peaks and troughs in the hydrographs.

(c) Quality of the 1-day annual maximum flood levels at selected gauging stations of the Brahmaputra were checked. The data was screened in the following three stages:

- Identification of the annual maximum 1-day water levels at a gauging site greater in magnitude than that of 1988 flood recorded at that station (known to be the highest observed). For example, the 1989-90 annual maximum flood level (28.35 m.PWD) recorded at Noonkhawa is higher than that of 1988-89 water level (28.10 m.PWD) recorded at the same station. Similar peak flood levels were also identified in the data of Chilmari and Kamarjani.

- Identification of the inconsistent peak flood level at a gauging site with respect to its upstream and downstream gauging station levels. eg. the 1-day annual maximum water level recorded at Kristomonichar in 1968-69 is higher than that recorded at Kholabarichar on the same day. There are a number of years when the 1-day annual maximum water levels recorded at Kristomonichar are higher than those recorded at Kholabarichar. The 1-day annual maximum water level data at Kristomonichar is considered to be of poor quality and was excluded from the peak water level analysis. Inconsistent peak flood levels of this type were also detected in the records of Kholabarichar, Kazipur, Jagannathganj and Teota.

- Detection of inconsistency in the range of variation of the 1-day annual maximum water level series at a gauging station. eg. the range of variation of the 1-day annual maximum water level magnitude is highest at Jognaichar. This wide range of variation in the peak water level series at Jognaichar is not consistent with the range of variations of its up-stream and down-stream gauging stations. This appears to be due to the unrealistic 1-day maximum water level recorded at Jognaichar in 1971-72.

2.5 Rainfall over the Catchment in Bangladesh

The Brahmaputra catchment area in Bangladesh is relatively very small and certainly insignificant in terms of the magnitude of flows in the river. On the right bank, the only substantial river flowing into the Brahmaputra within the study area is the Ghagot that drains the rather higher ground between Gaibandha and Rangpur and discharges through the Manas Regulator. Over the remainder of the study reach the drainage is generally away from the river, as reflected by the several old distributaries that were closed off when the BRE was constructed. These old channels still provide local drainage conveyance into the Karatoa-Bangali-Ichamati system that eventually drains into the Baral River and back into the Brahmaputra at the southern limit of the study area.

In order to obtain a general appreciation of the pattern of rainfall in the Brahmaputra valley, bounded on the west by the Barind Tract, five representative rainfall stations providing satisfactory areal coverage of the region were selected from a large network of daily recording stations for rainfall analysis (Figure 2.2).

The mean annual rainfall of the selected stations varies from 2,372 mm to 1,440 mm, with a clearly defined seasonal pattern as shown in Table 2.3. Approximately three-quarters of the annual rainfall is experienced during the monsoon period and the dry season rainfall is an insignificant fraction of the annual rainfall.

Table 2.3 Average Rainfall Pattern of Adjacent Areas

Season	Period	% of Annual
Pre-monsoon	April - May	16.6
Monsoon	June - September	72.2
Post-monsoon	October - November	8.4
Dry	December - March	2.8

Storm rainfall characteristics were investigated using the daily rainfall data of the selected stations. It was found that consecutive rainfall in the catchment area can be experienced for periods of up to 10 days during monsoon periods.

An analysis for the pre-monsoon (May-June) rainfall, the critical period for drainage design purpose, was carried out for all five selected stations. A frequency analysis of the annual maximum rainfall from one to ten day durations was undertaken and a Gumbel Extreme Value Type I distribution was fitted to the annual maximum series for each duration. The goodness-of-fit was checked by Kolmogorov-Smirnov test and by graphical plot. The results of the rainfall depth-duration-frequency analysis are given in Table 2.4.

The results show that relatively high storm rainfall can be experienced during the pre-monsoon season on the Brahmaputra catchment and its adjoining areas in Bangladesh and that this rainfall is not uniformly distributed in time and space over the catchment area. For example, the 1 in 2 design rainfall at Rangpur varies from 125 mm (1-day) to 294 mm (10-day) whereas, at Nachol, it varies from 68 mm (1-day) to 153 mm (10-day). Of the stations investigated, the rainfall intensity at Rangpur is the highest for storm duration of up to five days, whilst that at Jamalpur is the highest for storm duration greater than five days.

2.6 Discharge Characteristics

General Characteristics

The annual hydrograph of the Brahmaputra is characterised by low flows during the dry season associated with the winter months and high flows during the summer due to snowmelt in the Himalayas and heavy rainfall in the Assam Valley and Bangladesh itself, associated with the summer monsoon. In most years the Brahmaputra peaks in late July or early August and some overbank flooding occurs at this time. Mean peak flow is around 65,500 m^3/s, which corresponds closely to bankfull flow. The Ganges, in contrast, peaks in late August or early September (mean peak flow 51,625 m^3/s). On the few occasions when the Brahmaputra peaks late and coincides with the Ganges catastrophic flooding occurs that is far more extensive, prolonged and damaging than the normal seasonal inundation.

Table 2.4 Pre-monsoon Rainfall Depth-Duration-Frequency Analysis

Rainfall Station	Return Period (years)	Rainfall (mm) 1-day	2-day	3-day	5-day	7-day	10-day
Jamalpur (R-67)	2	101	149	174	212	242	285
	5	149	217	253	297	344	410
	10	181	261	305	354	412	493
	20	211	304	355	408	477	572
	25	221	318	371	425	498	597
	30	228	329	384	439	515	617
	50	250	360	420	478	562	674
	80% dep	66	99	116	148	165	193
Rangpur (R-206)	2	125	167	192	232	259	294
	5	175	232	264	312	348	390
	10	209	276	311	365	407	453
	20	241	317	356	415	463	514
	25	251	331	371	431	481	532
	30	260	341	383	444	495	548
	50	283	371	415	481	536	592
	80% dep	87	118	139	173	193	224
Bogra (R-6)	2	67	95	111	134	148	167
	5	104	152	178	217	242	268
	10	129	194	222	272	304	336
	20	153	232	264	325	364	400
	25	160	244	278	342	383	421
	30	166	253	289	355	398	437
	50	183	281	319	393	441	484
	80% dep	40	50	62	73	79	91
Nachol (R-190)	2	68	93	105	121	132	153
	5	98	137	153	180	197	229
	10	118	165	185	219	240	279
	20	137	193	215	256	282	327
	25	143	202	225	268	295	343
	30	148	209	233	277	306	355
	50	162	228	255	304	335	390
	80% dep	46	61	69	78	84	97
Pabna (R-25)	2	80	106	125	155	176	197
	5	110	144	171	209	236	261
	10	129	169	201	244	276	304
	20	148	193	231	278	315	345
	25	155	200	240	289	327	358
	30	159	206	248	297	337	369
	50	173	224	269	322	364	398
	80% dep	57	78	91	115	131	148

Average recorded daily flow of the Brahmaputra at Bahadurabad Transit (catchment area of about 501,000 km^2) is about 20,000 m^3/s, although the flow was less than this value for 60% of the gauging days during the period 1956-88. The mean annual flood flow at this station is about 65,500 m^3/s, while the highest recorded flood peak of 98,600 m^3/s was observed in 1988. It has been estimated that this peak flow has a return period of about 65 years, although it is only 4 percent less in magnitude than the nominal 100 year return period peak, based on the Bahadurabad data.

Discharge Measurement

The difficulties relating to the measurement of discharge in a large braided river such as the Brahmaputra are well appreciated by the BWDB. A variety of potential sources of error have been acknowledged, including micro changes in river morphology during measurement, which usually takes at least 48 hours; during rising and falling stage in particular, the bed may change its configuration substantially owing to migrating bedforms and under such conditions, surveys requiring periods longer than a few hours can result in unreliable measurements. Coleman (1969) commented that until the mechanics and patterns of bedform movement can be more thoroughly documented, any discharge measurement in the Brahmaputra may need to be treated as an estimate.

Definition of Discharge Characteristics

Thirty one years (1956-62, 1964-70, 1977-88) of data for Bahadurabad Transit were used to define the general discharge characteristics of the Brahmaputra. The general pattern of the discharge characteristics is illustrated in Figure 2.3. This shows that the discharge starts increasing in April and the first peak flood, generally occurring in July, is characterised by an extremely rapid increase in discharge. After this rapid initial rise the major peak occurs in August and the discharge begins to subside in October.

The highest and lowest 1-day discharges, estimated by BWDB at 98,300 m^3/s and 2,860 m^3/s, were observed in the months of August and March respectively. The within-the-month variation of discharge is the highest in August and the lowest in February. The former varies between 21,200 m^3/s and 98,300 m^3/s with a highly skewed distribution, while the latter ranges from 3,140 to 6,210 m^3/s and is almost symmetrically distributed about the mean.

Long-Term Trend in Peak Flood Flow Characteristics

Both the three year moving average and cumulative average methods were applied to investigate possible long-term trends. No significant trend was found in the peak flows of the Brahmaputra, Ganges or Teesta.

Peak Flood Discharge Analysis

In general, the Gumbel Extreme Value Type I distribution satisfactorily fitted the 1-day annual maximum discharge series of all the three rivers. The 100 year return period flood flows for the Brahmaputra, Ganges, and Teesta are estimated at 102,400 m^3/s, 87,400 m^3/s, and 10,300 m^3/s respectively. For comparison, the 1988 flood flow for the Brahmaputra had a return period estimated at 65 years whereas the 1988 flood flow for the Ganges had a recurrence interval of less than 20 years and that of the Teesta less than 10 years.

2.7 Analysis of Water Level Records

General Water Level Characteristics

Twenty nine years (1957-61, 1964-70, 1972-88) of screened mean daily water level data for the Bahadurabad Transit were used to define the general water level characteristics of the Brahmaputra. Figure 2.4 shows the same general pattern as for the discharge in Figure 2.3 but with substantially less difference between the maximum and 25 percent probability of exceedance values. The small variation in water level from year to year is illustrated by the fact that it was found that in general the difference between the 2 and 25 year return period water levels is of the order of only one metre.

Peak Water Level Analysis

In general, the Gumbel Extreme Value Type I distribution fitted the 1-day annual maximum water level series of the Brahmaputra at gauging stations where more than thirty years of data are available.

Water levels for different return periods for the 11 selected gauging stations of the Brahmaputra are given in Table 2.5 and the corresponding water level frequency curves for Noonkhawa, Chilmari, Bahadurabad Transit, Sirajganj and Teota are shown in Figure 2.5. These may be compared with those obtained from the 1-D modelling 25 year simulation runs shown in Figure 3.5.

Water surface profiles of the Brahmaputra for selected events are shown in Figure 3.4. The chainages for the gauging stations in Table 2.5 are based on the assumptions made for the schematisation of the Brahmaputra for the BRTS 1-D model (see Chapter 3 of this report). Location of the gauges for Chilmari, Bahadurabad Transit, Kazipur, Sirajganj, Mathura and Teota have been identified by BRTS on the ground and therefore the chainages for these gauges are considered to be reliable. Chainages corresponding to other gauges are only approximate.

2.8 Sediment Transport Relationships

Data Availability

Due to the scarcity of field data on bed load transport, the analysis was confined to suspended load. The suspended load is made up of wash load (defined in this case as material of grain size less than 0.054 mm) carried in suspension and suspended bed load of sand size which saltates and which plays an important part in the migration of the bed dunes (see Annex 1).

Historic suspended bed load data for the Brahmaputra prior to the BRTS was limited to 61 samples for the period 1968-70 and 200 samples for the period 1982-88. A total of 218 wash load samples were collected and analysed during the period 1971-88 but there were only 100 contemporaneous observations made for wash load and suspended bed load during the period 1982-88.

Quality checks of this data were carried out by visual inspection and by using graphical plots.

Table 2.5 Peak Flood Levels in the Brahmaputra

Station	Chainage (km)	No. of Sample	Return Period (Year) 2	4	5	19	50	100	1:100 FAP-25	1988 Flood Level
			Flood Level (m. PWD)							
Noonkhawa	0.0	27	27.49	27.85	27.98	28.06	28.13	28.16	29.20	28.10
Chilmari	34.0	23	23.96	24.30	24.53	24.74	25.02	25.23	25.30	25.06
Kamarjani	46.4	24	22.45	22.93	23.16	23.34	23.51	23.60	24.20	23.43
Kholabarichar	68.7	24	20.69	21.06	21.30	21.53	21.83	22.05		21.68
Bahadurabad (T)	76.6	32	19.76	20.07	20.27	20.47	20.72	20.91	20.70	20.62
Kazipur	135.0	21	15.57	15.92	16.16	16.38	16.67	16.89	16.40	16.77
Jagannathganj	139.0	21	15.38	15.73	15.94	16.13	16.37	16.53	16.40	16.14
Sirajganj	155.8	36	13.88	14.22	14.44	14.66	14.94	15.15	15.10	15.12
Porabari	188.2	36	12.13	12.55	12.83	13.10	13.44	13.70	13.10	13.15
Mathura	225.4	25	10.01	10.47	10.78	11.08	11.46	11.75	11.60	11.35
Teota	237.4	24	9.40	9.82	10.09	10.35	10.69	10.94	10.70	10.58

Notes:

1. The chainages of the gauging stations are based on the schematisation of the Brahmaputra for the BRTS 1-D model.
2. The chainages of the gauges are approximate excepting the location of the gauges at Chilmari, Bahadurabad Transit, Kazipur, Sirajganj, Mathura and Teota which have been identified by the Consultants on the ground.
3. Estimate of the 1 in 100 year water level made by FAP-25 is preliminary.

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Suspended Sediment Transport Characteristics

Figure 2.6 indicates that the monthly suspended bed load transportation of the Brahmaputra is highest during the period July to September, thereafter dropping off to a relatively minor level from November through to April. During the month of July it is likely to exceed 18 million tonnes.

Wash load follows the same general pattern but with a very much more pronounced increase during the July to September period. The peak level is reported to be in September, reaching almost 200 million tonnes in the month; if correct, this peaking of wash load after the flood peak, implying higher concentrations later in the monsoon season, is somewhat out of the ordinary.

Suspended Sediment Rating Curve

In this study an attempt was made to establish relations between suspended bed load transport, wash load transport and total suspended sediment discharge with the mean daily streamflow at Bahadurabad Transit from the available historic data. A least square linear regression model was used for this purpose and for the suspended load the regression equation provided a satisfactory fit to the observed data.

Seasonal and annual variation of the relationship was then investigated. It was found that the 1968-70 and 1982-88 relationships between the suspended bed load and the mean daily streamflow exhibit slightly different characteristics to that of their overall relationship (see Figure 2.7). The co-efficient of determination for the 1968-70 data was 0.86 whereas the co-efficient of determination for the 1982-88 data was 0.89 but when the two sets were considered together the co-efficient of determination dropped to 0.795.

A similar relationship was established between the wash load and the mean daily flow (see Figure 2.8) but in this case the correlation was poor, with a coefficient of determination of only 0.68.

Can we address the dynamic process with our present tools

3. MATHEMATICAL MODELLING

3.1 Objectives and Scope

The various forms of mathematical modelling utilised in this study have shown themselves to be valuable working tools for analysing time dependent dynamic processes that cannot readily be addressed by less sophisticated techniques. It remains important however that the role of mathematical modelling be well defined and regularly reviewed in order to ensure that it is closely integrated with other components. At all times, it is important to relate the form, resolution and accuracy of the model systems to both the data available and the dictates of the end requirements in the form of output and to be able to quantify the errors inherent in the analysis, particularly those due to data limitations. → what about error due to not taking morphological factors due to various limitations

The four main forms in which mathematical modelling contributed to this study have been:

(a) 1-D hydrodynamic modelling of the Brahmaputra (Jamuna) River, using a refined version of the SWMC General Model, which has been used to provide design water levels for the short-term works, boundary conditions for the 2-D and physical models, water level and discharge data for the morphological studies and as a tool for assessing the hydrodynamic impact of alternative containment strategies. It has also been used to quantify the probability distribution of flow velocities below certain values as a means of determining reliable construction windows for river works.

(b) 1-D hydrodynamic modelling of the Karatoya-Bangali-Ichamati river system has been used to quantify the change in flooding regimes consequent upon breaches in the BRE.

(c) 2-D morphological modelling, which implicitly includes 2-D hydrodynamic modelling, to gain insight into, and to quantify, the morphological processes that are associated with scour development and bank erosion and to investigate the viability of alternative means of influencing channel development. This part of the study has been carried out in close coordination with the physical modelling, the two approaches complementing each other.

how far we have been able to quantify the morphological

(d) 1-D morphological modelling has contributed to the assessment of the magnitude of long term channel geometry changes that are associated with the alternative levels of river containment being considered.

The scope of the mathematical modelling component was set down in the BRTS Working Paper on Mathematical Modelling (December 1990) together with the details of the model systems being employed. The former has changed only in detail and emphasis since then and there has been no substantial change in the latter. The calibration and verification of the modelling systems, the scope of the applications and the interpretation of the results will be found described in greater detail in the separately published BRTS Report on Model Studies.

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3.2 1-D Modelling of the Brahmaputra (Jamuna) River

3.2.1 Model Characteristics

A considerable amount of time and effort was expended in setting up the refined version of the SWMC General Model in order that it should have the resolution and data quality appropriate to the Study's requirements. The extent of this model is shown in schematic form in Figure 3.1.

Since the intensity of cross-section data is the single most important element when seeking improved resolution, the model was first calibrated using the 1986/87 cross-sections, the largest set of contemporaneous sections available (see Figure 3.2). Subsequently, during the refinement process, it became apparent that data quality was also a significant factor and time and resources were diverted to address this aspect.

Other than the cross-sections, the most sensitive data are the boundary conditions and the water level records. There is only limited scope for refining the boundary conditions and the values derived by the SWMC have in general been adopted; also the approach recommended by FAP-25 was consistently followed. Similarly, analysis of the Brahmaputra water level records by both BRTS and FAP-25 led to the conclusion that the records are in general internally consistent, the main source of uncertainty being generally in the vertical and horizontal control for the gauge posts.

Cross-section data was compared visually with the satellite imagery and found to be generally consistent in form but only if the section lines were shifted from those indicated on the key plan. This led to the field checking of a sample selection of the cross-section reference pillars with the result that the positions of several had to be adjusted (see Second Interim Report Annex 2 for full details). The location of gauging stations was also checked in the field and, in the worst case, a difference of 10 km was identified. This shift resulted in some very significant changes to the water surface profile in the vicinity of Sirajganj and the elimination of a feature that was initially thought to be morphologically significant.

The calibration of the model was completed after fine adjustment of the roughness values within prescribed limits. It was found that satisfactory simulation could be obtained with M values in the range 31 to 50 for bankfull conditions and 20 to 40 for low flow. Since the match of water levels at Bahadurabad, one of the two gauging stations on the left bank, was less good, a sensitivity test was carried out to determine whether this could be explained by potential error in the flow gaugings, which earlier analysis (see First Interim Report), showed could be of the order of 20 percent, with the probability being that the Bahadurabad values tend to be overestimated. It was found that a reduction of simulated flow of only 15 percent would explain the difference, illustrating the careful interpretation that is required when calibrating.

The verification of the model using the 1989/90 cross-sections gave results that were consistent within the limits identified as potential error ranges attributable to both data and the dynamics of the river. It has been noted that although the river has all the signs of being in dynamic equilibrium overall, the cross-sectional properties at-a-site vary significantly as large sand bars move down the river (see Annex 2). This is independent of any larger scale slug flow of sediment arising from external changes that may be affecting sediment inflow rates.

Thus since the 1988/89 set of cross-sections contains only 31 sections, compared with the 61 of 1986/87, it is to be expected that the "noise" will be relatively more pronounced and the fit between simulated and predicted water levels less good (see BRTS Second Interim Report Annex 2 for further details).

3.2.2 Model Application: 25 Year Series Runs

As recommended by FAP-25, estimation of design water levels for future development schemes were derived from a statistical analysis simulated water levels using 25 years of historical hydrographical data (1965-89). Three 25 year simulations were made using the BRTS 1-D model for the Brahmaputra River. The first 25 year model run with current BRE location and left bank condition was made after final calibration and verification had been completed to the standards set by FAP-25.

The earlier runs carried out by FAP-25 showed a good match between simulated and observed water levels over the 25 year period despite the fact that the model used fixed cross-sections relating to only one of these years. Since the BRTS model contained a number of refinements and operated at a higher resolution a further verification was carried out to confirm that the match remained satisfactory.

Simulation results from the 25-year model run using existing BRE location and current Brahmaputra left bank condition were compared with BWDB observed annual peak water levels at the reference gauging stations used by BRTS for calibration of the 1-D model. Brahmaputra comparison stations include Chilmari, Bahadurabad Transit, Mathurapara, Kazipur, Sirajganj, Mathura and Teota. Except for Bahadurabad Transit and Teota all gauges are located on the Brahmaputra right bank. Teota is located near the Brahmaputra Ganges confluence at Aricha and hence directly influenced by Ganges flow.

As shown in Figure 3.3 and Table 3.1 the average variance between 25-year observed and modelled annual peak water levels at Brahmaputra right bank reference gauging stations is generally satisfactory. Average departures for the 25-year period at these stations lie within 0.10 m. However model performance for the 25-year period is less good at the gauging station located on the Brahmaputra left bank. This could arise from various combinations of datum and flow gauging errors. Since the BRTS is concerned with the right bank of the river, it was agreed with FAP-25 that it would be correct in this instance to give greater weighting to matching the right bank gauge records.

Three sub-models were used to investigate the impact of different degrees of intervention and confinement on the magnitude of velocity and water level along the river for different return periods.

The Base Model was the full calibration model with the Teesta simplified to a lateral inflow to the main river. This modification improved model performance without any loss of output quality.

The second sub-model included the constriction and confinement associated with the construction of the Jamuna Multi-purpose Bridge. Since the final length of the bridge remained to be finalised, three different openings between the abutments were modelled. The upper value of 5,600 m is close to the mean bankfull width of a single channel section of the river

Table 3.1 Source of 1965-89 Boundary Data for 25 Year Simulation Using 1-D Model of the Brahmaputra River

River	Gauging Station	Type of Boundary	Source
Brahmaputra	Bahadurabad Transit	Flow	FAP-25 (corrected and revised using BWDB measured data
Teesta	Kaunia	Flow	FAP-25 (Data base created using BWDB data)
Ganges	Hardinge Bridge	Flow	FAP-25 (Simulation output from Run 5)
Meghna	Bhairab Bazar	Flow	FAP-25 (Simulation output from Run 5)
Gorai	Gorai Railway Bridge	Water Level	FAP-25 (Simulation output from Run 5)
Arial Khan	Madaripur	Water Level	FAP-25 (Simulation output from Run 5)
Meghna	Chandpur	Water Level	FAP-25 (Simulation output from Run 5)

as computed using the relationship proposed in the JMB Design Report and independently verified by the BRTS. This model run is also of relevance for the assessment of the impact of node stabilisation measures that could form part of a long-term plan for management of the river.

The third sub-model was designed to study the impact of the construction of a near continuous left bank flood embankment in accordance with the proposal prepared by FAP-3. The main impact of this proposal would be the closing of the main Brahmaputra flood plain spillage channel (ARJAM as shown on Figure 3.1) and its four linkage channels to the Brahmaputra at nodes ADR3, ARD4, ARD5 and ARD6. The Old Brahmaputra and the lower Dhaleswari channels would be unaffected.

3.2.3 Model Application: Single Severe Event Runs

Running the full 25 year cycle for all permutations and combinations of intervention possibilities would have been impracticable and of limited value, since the main objective is to compare the maximum water level and mean velocity that occurs for each scenario. It was agreed that the most appropriate approach would be to run the model using boundary conditions based on a known severe hydrological event that was of the same order of magnitude as the design event (100 year return period). The 1988 flood event fits this criterion and was accordingly used for this purpose.

In all fourteen sub-models were set up and run in this way. The variables investigated being the distance that the BRE is set-back from the bank of the river, the width of the confinement associated with the construction of the Jamuna Bridge and the degree of confinement on the left bank.

In addition to the present position of the BRE tested in the base case, simulations were made with the embankment set back by 2 km and 4 km. The first of these values represents the approximate average set back of the original BRE when constructed and the second is a more extreme example for comparison purposes.

The variations of the width of opening after construction of the Jamuna Bridge was taken as the same as used for the 25 year series runs.

Two different degrees of left bank confinement were investigated. The first corresponded to the extension of the left flood embankment in accordance with the proposal prepared by FAP-3 and was the same as used for the 25 year series. The second was a hypothetical extreme confinement used in order to provide a bounding value for impact quantification purposes. In this case in addition to the FAP-3 confinement, all left bank distributaries, including the Old Brahmaputra were simulated as closed off.

3.2.4 Results of the 25 Year Series Model Runs

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The longitudinal profile of the water level for 100 year return period, bankfull and dominant discharge is shown in Figure 3.4. The steeper gradient above the Teesta confluence and the backwater effect of the Ganges extending back to around 50 to 70 km upstream from the Ganges confluence (between Betil and Sirajganj can also be seen.

Figure 3.5 illustrates the impact on water level, for different return periods, as a result of constructing the Jamuna Bridge (4,608 m water opening) and thereafter the extension of the left bank flood embankment.

Did the analysis took of morphological effect due to bridge construction + left embankment

The impact of Jamuna Bridge construction is most marked over the 20 km immediately upstream, for example at Sirajganj, 6 km upstream, where the water level is increased by 45 cm for a 100 year return period. By Sariakandi, 55 km upstream of the bridge, the effect has virtually disappeared. Extension of the left bank flood embankment would have a greater effect, extending over the whole study reach, though diminishing to minimal at the confluence with Teesta. Again, the effect is most conspicuous in the Sirajganj area, with a rise of 70 cm above the "with Jamuna Bridge" water level for a 100 year return period.

Figure 3.6 provides a similar comparison of the impact on mean velocity at representative cross-sections, from which the effect of the reduction in area of the left bank flood plain available for flood attenuation can be seen in the relatively high flow which results downstream.

3.2.5 Results of the Single Severe Event Runs

The full set of results in terms of both water level and mean velocity is presented in matrix form in Tables 3.2 and 3.3. The model run index is given in Table 3.4. The results are illustrated graphically in Figures 3.7 to 3.9, which show the change in peak water level for the simulated 1988 event as compared with the base case representing the present condition.

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Table 3.2: Water Level of Selected Return Periods Obtained by Analysing Simulated Annual Maximum Water Levels for the 25 Year Period

Brahmaputra 1-D Model Chainage (Km)	25 Year Model Run No.	Return Period (Year) 2	5	10	25	50	100	250	500	1000	2500	5000	10000
44.25	Run-3	23.130	23.624	23.902	24.215	24.426	24.622	24.865	25.039	25.207	25.420	25.576	25.728
	Run-4	23.133	23.628	23.907	24.220	24.432	24.629	24.873	25.048	25.216	25.430	25.587	25.740
	Run-5	23.156	23.657	23.940	24.258	24.472	24.672	24.919	25.096	25.267	25.484	25.643	25.798
63.30	Run-3	21.592	22.033	22.283	22.565	22.756	22.934	23.155	23.313	23.466	23.660	23.803	23.942
	Run-4	21.595	22.038	22.290	22.573	22.765	22.944	23.165	23.324	23.478	23.673	23.817	23.957
	Run-5	21.651	22.108	22.366	22.656	22.853	23.036	23.264	23.427	23.584	23.784	23.931	24.074
75.60	Run-3	20.657	21.098	21.346	21.622	21.808	21.980	22.193	22.345	22.491	22.676	22.812	22.943
	Run-4	20.659	21.103	21.352	21.630	21.818	21.992	22.206	22.360	22.507	22.694	22.831	22.964
	Run-5	20.764	21.227	21.486	21.776	21.970	22.151	22.373	22.532	22.684	22.878	23.019	23.156
84.15	Run-3	20.220	20.663	20.912	21.191	21.379	21.553	21.769	21.923	22.071	22.259	22.397	22.532
	Run-4	20.228	20.671	20.919	21.197	21.385	21.558	21.773	21.926	22.073	22.260	22.397	22.530
	Run-5	20.364	20.832	21.094	21.388	21.585	21.768	21.994	22.155	22.310	22.507	22.651	22.791
113.40	Run-3	18.149	18.572	18.809	19.072	19.249	19.412	19.614	19.757	19.895	20.069	20.196	20.320
	Run-4	18.169	18.597	18.836	19.109	19.281	19.446	19.649	19.794	19.932	20.108	20.236	20.361
	Run-5	18.468	18.940	19.201	19.490	19.683	19.860	20.078	20.233	20.380	20.567	20.703	20.834
136.80	Run-3	16.205	16.604	16.824	17.068	17.231	17.381	17.565	17.696	17.821	17.979	18.094	18.206
	Run-4	16.270	16.685	16.915	17.170	17.341	17.499	17.692	17.829	17.961	18.127	18.248	18.366
	Run-5	16.669	17.146	17.410	17.704	17.901	18.083	18.306	18.465	18.617	18.810	18.950	19.086
151.75	Run-3	14.674	15.058	15.273	15.513	15.674	15.824	16.008	16.139	16.265	16.425	16.542	16.655
	Run-4	14.846	15.267	15.502	15.765	15.942	16.106	16.308	16.452	16.590	16.766	16.894	17.019
	Run-5	15.309	15.806	16.085	16.397	16.607	16.801	17.042	17.213	17.378	17.588	17.741	17.890
162.35	Run-3	13.936	14.319	14.532	14.771	14.931	15.079	15.261	15.392	15.516	15.675	15.790	15.902
	Run-4	14.204	14.637	14.880	15.151	15.334	15.504	15.713	15.862	16.006	16.188	16.321	16.451
	Run-5	14.676	15.188	15.476	15.800	16.018	16.220	16.470	16.649	16.822	17.041	17.201	17.357
170.75	Run-3	13.346	13.716	13.924	14.156	14.312	14.457	14.636	14.765	14.888	15.044	15.158	15.269
	Run-4	13.399	13.788	14.006	14.250	14.414	14.566	14.754	14.889	15.018	15.181	15.301	15.418
	Run-5	13.820	14.283	14.546	14.843	15.044	15.232	15.465	15.633	15.795	16.002	16.153	16.302
180.60	Run-3	12.307	12.689	12.913	13.170	13.348	13.517	13.728	13.883	14.033	14.227	14.370	14.512
	Run-4	12.341	12.732	12.960	13.223	13.405	13.576	13.791	13.948	14.101	14.297	14.443	14.587
	Run-5	12.760	13.229	13.505	13.824	14.044	14.254	14.518	14.710	14.898	15.141	15.321	15.499
185.70	Run-3	11.878	12.259	12.485	12.747	12.930	13.103	13.323	13.484	13.641	13.844	13.996	14.146
	Run-4	11.911	12.303	12.534	12.803	12.989	13.166	13.389	13.552	13.712	13.918	14.072	14.224
	Run-5	12.311	12.777	13.054	13.378	13.605	13.820	14.094	14.295	14.492	14.748	14.939	15.129
205.15	Run-3	10.404	10.848	11.148	11.532	11.822	12.116	12.511	12.818	13.133	13.562	13.897	14.241
	Run-4	10.433	10.886	11.189	11.574	11.862	12.151	12.539	12.839	13.144	13.559	13.881	14.211
	Run-5	10.804	11.320	11.651	12.060	12.360	12.656	13.046	13.341	13.639	14.037	14.343	14.653

Notes: 25 Year Simulation Option

Run-3 – Present BRE Location + Current Brahmaputra Left Bank Condition

Run-4 – Present BRE Location + Current Brahmaputra Left Bank Condition + JMB (4,608 m)

Run-5 – Present BRE Location + FAP-3 Brahmaputra Left Bank Condition + JMB (4,608 m)

All water level in m.PWD.

Frequency analysis of water levels for different project scenarios were done by fitting a Log Normal distribution to the sample data. Parameters of the distribution were estimated by modified maximum likelihood method. This was acheived by using HYMOS software at FAP-25.

Revised on 23-12-1992

Table 3.3: Velocity of Different Return Periods at Selected Chainages of Brahmaputra River under Various Engineering Schemes

Brahmaputra 1-D Model Chainage (Km)	25 Year Model Run No.	Return Period (Year) 2	5	10	20	50	100	200	Remarks
44.25	Run-3	0.991	1.034	1.063	1.091	1.127	1.154	1.181	Downstream of
	Run-4	0.991	1.034	1.063	1.091	1.127	1.154	1.181	Teesta
	Run-5	0.988	1.029	1.057	1.083	1.117	1.143	1.168	Confluence
63.30	Run-3	1.743	1.792	1.824	1.856	1.896	1.926	1.957	Manas
	Run-4	1.742	1.791	1.824	1.855	1.896	1.926	1.957	Regulator
	Run-5	1.721	1.766	1.795	1.823	1.860	1.887	1.914	
75.60	Run-3	1.059	1.096	1.121	1.144	1.175	1.198	1.221	Fulchari
	Run-4	1.059	1.095	1.119	1.141	1.171	1.193	1.215	
	Run-5	1.035	1.066	1.087	1.107	1.133	1.152	1.172	
84.15	Run-3	1.202	1.245	1.274	1.301	1.337	1.363	1.390	
	Run-4	1.200	1.244	1.273	1.300	1.336	1.363	1.390	
	Run-5	1.164	1.202	1.228	1.252	1.283	1.307	1.330	
113.40	Run-3	1.434	1.453	1.465	1.478	1.493	1.505	1.516	Mathurapara
	Run-4	1.427	1.444	1.455	1.465	1.478	1.489	1.499	
	Run-5	1.419	1.436	1.447	1.458	1.472	1.482	1.492	
136.80	Run-3	1.200	1.222	1.236	1.250	1.268	1.281	1.294	Kazipur
	Run-4	1.182	1.199	1.211	1.222	1.236	1.247	1.258	
	Run-5	1.149	1.165	1.175	1.184	1.197	1.206	1.216	
151.75	Run-3	0.994	1.020	1.037	1.053	1.074	1.090	1.106	Simla
	Run-4	0.953	0.971	0.983	0.994	1.009	1.020	1.031	
	Run-5	0.970	0.995	1.012	1.028	1.049	1.064	1.079	
162.35	Run-3	1.386	1.404	1.415	1.426	1.440	1.451	1.462	Sirajganj
	Run-4	1.315	1.325	1.332	1.339	1.347	1.353	1.360	
	Run-5	1.318	1.328	1.335	1.342	1.351	1.357	1.364	
170.75	Run-3	1.227	1.242	1.251	1.261	1.273	1.282	1.290	Downstream edge
	Run-4	1.686	1.804	1.882	1.957	2.055	2.127	2.200	of JMB Guide
	Run-5	1.805	1.946	2.039	2.129	2.245	2.332	2.418	Bunds
180.60	Run-3	1.085	1.106	1.120	1.134	1.151	1.164	1.177	Belkuchi
	Run-4	1.085	1.107	1.122	1.136	1.154	1.168	1.181	
	Run-5	1.117	1.148	1.168	1.188	1.213	1.232	1.251	
185.70	Run-3	1.720	1.760	1.787	1.812	1.845	1.870	1.894	Betil
	Run-4	1.723	1.763	1.789	1.815	1.848	1.873	1.897	
	Run-5	1.760	1.805	1.835	1.864	1.901	1.929	1.956	
205.15	Run-3	1.542	1.601	1.621	1.634	1.643	1.647	1.650	
	Run-4	1.545	1.605	1.626	1.639	1.649	1.654	1.657	
	Run-5	1.611	1.663	1.717	1.769	1.837	1.888	1.939	

Notes: 25 Year Simulation Option:

Run-3 - Present BRE Location + Current Brahmaputra Left Bank Condition

Run-4 - Present BRE Location + Current Brahmaputra Left Bank Condition + JMB (4,608 m)

Run-5 - Present BRE Location + FAP-3 Brahmaputra Left Bank Condition + JMB (4,608 m)

All velocities are in m/s.

The increase in upstream water levels resulting from the construction of Jamuna Bridge is evident in all three figures, the narrower the span the higher the levels (Figure 3.9). The reduction in water levels according to setback distance of the BRE is shown in figures 3.7 and 3.8, and the increase in levels - one metre at the bridge site, Figure 3.8 - indicates the very significant effect which would be caused by confinement of the left bank.

3.2.6 Construction Window Estimation

A key consideration when planning the construction of river training works that involves underwater work is the "construction window" that can be expected. The window will be determined by the number of days during which water levels and velocities are below set limits; for instance it would be impracticable to attempt to lay geotextile fabric under water when current velocities exceed 1.5 m/s without highly specialised plant and equipment.

The simulated the water levels and average current velocities at each cross-section of the 1-D hydrodynamic model for the period 1965-1990 were used for this purpose. Although average velocities were not saved during the MIKE 11 run, it was possible to derive this information from an analysis of water level, flow data and cross-sectional data.

The results of this analysis have been produced as a series of cumulative probability plots as illustrated in Figure 3.10 Included on the plots is also the average velocity which corresponds to each water level.

what is the ratio?

The velocities which occur adjacent to the construction works will be greater that the average velocity at that point in the river. The ratio of near bank velocity to average velocity estimated as part of the physical modelling programme has been applied to obtain representative values for the sites. When using these curves it is important to make due allowance for the fact that the average velocities derived from this work are based upon only one year of cross section data and so may not represent the worst case that can occur at a particular site.

3.2.7 Frequency of Char Inundation

A similar analysis to that used for the computation of construction windows was used to investigate the frequency of inundation of chars for different depths and durations, both for the present situation and after the construction of a local constriction to the channel such as the Jamuna Bridge. The results for two locations, Kazipur and Sirajganj are illustrated in Figure 3.11. As would be expected, the impact at Kazipur is relatively minor. It should be emphasised that these changes represent the condition immediately after construction and before bed adjustment in the vicinity of the bridge has taken place; after one, or at most two seasons, the impact will be much reduced

3.3 1-D Modelling of the Right Bank Flood Plain

This work was carried out in close cooperation with FAP-2 who have some overlap of interests. The basic model as taken over from the SWMC was an early version that did not at that time have any flood plain data appended to the sections, although the SWMC had a short time earlier completed the digitisation of the topographical data and processed this into stage area curves for the flood plains. The task of appending this new data to the cross-sectional data turned out to be substantially more difficult than first envisaged due to elevation

Fig? 80

differences of up to 4 m between the two data sets. By reference to the 4 inch and 8 inch to a mile mapping and to other topographical data obtained as a part of the study it was possible to rectify these mismatches, although at a significant cost in terms of both elapsed time and man-hours. The schematic for the completed model is shown in Figure 3.12.

A joint field trip with FAP-2 modellers during August 1991 provided a very good opportunity to observe the flows through the Mathurapara and Simla (Sonali Bazar) breaches in the BRE and to follow these through into the river system at a time when first hand data could be collected. Use was made of the recently available GPS position fixing system, which can easily and quickly be transferred from boat to jeep or rickshaw, to provide horizontal control.

Calibration of the model was satisfactory within the limits set by the availability of data and the uncertainty over actual flow conditions in particular years. Subsequently it was possible to compare the simulated areas of inundation with the visual evidence of the August 1987 satellite imagery processed by ISPAN under the FAP-19 component, which provided very satisfactory verification. See plate 14.

After some trials, it was decided that the most appropriate approach to modelling the breaches was to assume that the breach itself would not act as a significant control over the time period that was of relevance, since the erodible bank material would allow the width of the breach to increase until velocities were quite low. The control was then imposed by means of a wide shallow channel scaled on the topography obtained by inspection of the imagery and aerial photography and the topographic maps. This gave realistic simulations and is considered to be a suitable approximation of the actual conditions for the objectives of this particular analysis.

Breaches were simulated at Fulchari, Sariakandi/Mathurapara, Kazipur, Simla (Sonali Bazar), south of Sirajganj and Betil. This gave a good range of conditions varying from the virtually direct connection between Brahmaputra and Bangali at Sariakandi/Mathurapara (Figure 3.13) and the substantial overland distance traversed by flows through a breach in the vicinity of Fulchari. The significance of the breach in terms of the distance upstream was also clearly shown with the increased flood inundation depth and duration due to the breach at Betil being almost negligible in comparison to that due to breaches further north.

Breach flows as simulated were close to those estimated during the field visits and consistent with those earlier implied by the FAP-2 modellers from other indications. The Mathurapara breach flow as simulated peaked at 1,500 m^3/s with 1,200 m^3/s being observed in 1991. At the other end of the range the predicted flow through a breach at Betil was 475 m^3/s. The values for other locations are shown in Table 3.4.

Table 3.4 Maximum Modelled Flow Spilled into the Ghagot-Karatoya-Bangali System through Breaches in BRE

Breach Location	Maximum Flow Spilled (m^3/s)
Fulchari	950
Mathurapra	1,500
Kazipur	1,400
Sonali Bazar	900
South of Sirajgnaj	750
Betil	750

In spite of the limited time series data available (1986 to 1989), the results of the simulation model did permit the derivation of crop damage frequency curves, and thereby the establishment of "expected" annual crop losses priority location. However, the crop loss estimates obtained were regarded as a sufficiently accurate representation of an typical year for the purpose of the economic analysis.

The value of the model simulation was that it was possible to make an objective comparison of not only the depth of flooding but also the duration and timing. this information was then converted into changes in flood regime and thence to production loss. The incremental areas affected varied very considerably from less than 500 ha due a breach at Betil, an area that is strongly influenced by backwater effects up the Hurasagar river, to 25,000 ha in the case of a breach at Sariakandi or Mathurapara, which are situated relatively far north in a relation to the river system and where flows can enter almost directly into the Bangali river. The equivalent estimated net production losses ranged from Tk 3.5 million to Tk 225 million.

3.4 1-D Morphological Modelling

3.4.1 Background and Objectives

It may be anticipated that measures currently being considered for flood alleviation along the Brahmaputra, and for bank protection and river training, will have some influence on the future morphology (slope and geometry) of the river channel(s). The purpose of the 1-D Morphological Model is to help assess the magnitude and timescale of changes that may occur to the natural state of the river as a result of engineering works on the floodplain and along its banks.

The 1-D morphological modelling was therefore directed at determining:

(a) The order of magnitude of the changes to the river morphology likely to result from flood alleviation and river training measures

(b) The timescale over which such changes may occur

(c) The sensitivity of (a) and (b) to changes in boundary conditions upstream and downstream of the BRTS study area.

3.4.2 Review of Recent Studies

Two recent studies that have published results of analyses which are particularly relevant to the BRTS analysis of river morphology were reviewed. These studies are: the Jamuna Bridge Project, Phase 2 Study (1989 - 1990), and the study of Flood Control and River Training on the Brahmaputra River in Bangladesh carried out by a China - Bangladesh Joint Expert Team from 1989 to 1991. In both studies an assessment was made of general erosion and accretion down the Brahmaputra which could result from various engineering interventions along the course of the river as well as changes in boundary conditions upstream and downstream of the Brahmaputra (Jamuna) in Bangladesh. Both studies used 1-D morphological models in their analysis.The results from these studies are summarised below.

The Jamuna Bridge Project study was primarily concerned with the impact of the bridge construction on the river bed upstream. The river was assumed to be initially in equilibrium and calibration comprised adjusting the boundary conditions to the model so that equilibrium was reproduced. The assumption of channel equilibrium was based on the conclusions, from the study of the stability of the river, by Klaassen and Vermeer (1988).

A series of changes in the upstream and downstream boundary conditions was assumed to take place in full at the outset of the 75 year duration simulation period. Thus the results (summarised in Table 3.5) can be regarded as upper limit changes.

Table 3.5 Water Level and Bed Level Change at Sirajganj (after JMBA, 1990)

Boundary Change	Change in Level at Bed	Sirajganj (m) Water Surface
(a) Increased sediment inflow	0.2	0.1
(b) Reduced sediment inflow	-0.5	-0.3
(c) Increase in dominant discharge	-1.0	-0.8
(d) Increase in downstream water level	0.9	-0.7
(e) Abstraction of water near the border with India	0.9	0.7

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The following qualifications are noted in the report:

- Changes in sediment inflow propagate slowly through the system whereas changes in (dominant) discharge or the downstream boundary have a more rapid effect on bed degradation or aggradation
- the changes in bed level are in the model confined to the river channel; in practice aggradation of the bed will cause a rise in level of the flood plain and the chars, and this sedimentation over a wider area will reduce the rate of change indicated by the model.

None of the above scenarios considered a reduction in the overall channel width, as might be a consequence of river training. This issue is focussed on by the China - Bangladesh Joint Expert Team (CBJET).

The CBJET applied two types of 1-D morphological model to the Brahmaputra (Jamuna) between survey sections J-17 and J-1.

The first (referred to as the `hydrological hydrodynamic model) considered the flow and sediment exchange between the river channels and the chars and flood plain. An equivalent compound channel comprising a main channel plus floodplain was constructed from the surveyed sections of the Brahmaputra. The river was split two reaches, one upstream of Bahadurabad, the other from Bahadurabad to the confluence with the Ganges.

The model was calibrated as to reproduce the changes in weight of sediment due to the overall reported degradation of the channel during the period 1965 - 81 and the overall aggradation of the channel reported for the period 1982 - 89. The computed changes are between 64 percent and 68 percent of the measured changes, the measurements having been made from the analysis of historic channel cross-sections. The validity of this approach must be viewed in the light of the quality of the cross-section data that is discussed in Chapter 6 of this report.

A second model, called the `Modelof Non-Equilibrium Transport of Sediment with Non-uniform Grain size' was set up to assess the effect on the prediction from the first model of changes in the size distribution of the suspended sediment load at Bahadurabad for the two periods 1965/66 to 1980/81 and 1980/81 to 1988/89.

The analysis comprised a forecast of natural river changes over a 25 year period followed by an assessment of the effects of the implementation of a river training scheme combined, variously, with:

(a) an increase in the sediment inflow of 20 percent

(b) a rise in the water level at the confluence of 0.5 m

(c) flood embankments placed 5 km, 20 km and 50 km apart.

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It is important to note that the model predicts that natural changes occurring as a result of the persistence of current conditions in the Brahmaputra (Jamuna) over the next 25 years will result in overall deposition. However underlying this trend there will be erosion of the river channel upstream of Section J-8 (5 km north of Simla) leading to a lowering of the main channel by 0.1 to 0.3m (causing a lowering of water level at bank-full discharge of 0.2m). For the river downstream of Section J-8, deposition of up to 1.0m in the main channel (accompanied by a rise in water level of 0.6m) is reported.

Reasons need to be discussed

The reported results indicate that the overall effect of river training works would be erosion of the main channel and deposition on the flood plain following an initial rise in "flood" water levels of 0.5m in the first ten years after the completion of river training. Twenty five years after the completion of the river training schemes, water levels in the main channel would drop by 0.4m to 1.0m as compared with the present (natural) situation. Although the results of the two models are "consistent qualitatively" the second model (the "Non-equilibrium Transport" model) shows the deeper of the two erosion limits of 1.0m after 25 years, with the first model predicting slight aggradation (relative to the natural changes) downstream of J8.

An increase in in flowing sediment load (of 20 percent), doubling in the mean sediment grain sizes and in a rise in water level of 0.5m at the confluence with the Ganges are all described as having "unfavourable effects" on the channel upstream of the confluence. Each of these process, taken separately would increase the deposition in the main channel and raise water levels by about 0.5m over a 25 years period.

Reducing the spacing between flood embankments from 50 km to 20 km was shown to increase water levels by 0.1m. A further reduction to 5 km would reportedly cause a rise in water level of 0.4m but this would be accompanied by increased channel erosion, as a result of the implementation of river training works.

3.4.3 Selection of Sediment Transport Model Parameters for Current Study

Sediment Transport Formula

The Engelund-Hansen formula was selected as the most appropriate for sediment transport computations essentially because the available data were not sufficiently detailed to justify the use of a more complicated formula.

Bed Material Grain Size

All sediment transport models are sensitive to specification of grain size.A representative bed material grain size needs to be specified at each channel section as input data to the Mike11 model. Bed material samples taken at Test Areas 1 and 2 show both a seasonal variation at a site (d_{50} varies from 0.15 mm to 0.19 mm between July and November 1990 at Test Area 1), and to a lesser extent, a variation between the two sites (a d_{50} of 0.17 mm was measured at Test Area 2 in August 1991). Lates (1988) discusses the JICA bed material grading curves taken at Chilmari, Nakfater Char and Nagarbari in 1969. Although these show little variation in mean grain size (d_{50}) with progression downstream, the seasonal variation observed at Chilmari is significantly greater than that observed at Nagarbari. Possible causes of seasonal and spatial variation in bed material grain size are discussed by Goswami (1985), Bristow (1988) Lates (1990) and in Annex 1 of the BRTS First Interim Report (1991).

Previous studies have applied either a constant representative grain size to the Brahmaputra(Jamuna) between the border with India and the Ganges confluence (eg d_{50} of 0.16 mm was used in the JMBA analysis), or a reduction in size uniformally down the river. The CBJET assumed a grain size decrease from 0.26 mm to 0.17 mm along the Jamuna river inside Bangladesh. Since there does not appear to be sufficient evidence to support this upper value, whereas there are definite indications of a variation in grain size, the assumption made for this present study is that the grain size decreases from 0.21 mm at the Indian border to 0.16 mm at Aricha.

Calibration

For calibration purposes some assumptions have to be made about the present sediment transport balance in the river in its natural state. Exhaustive analysis of topographic data from the last 25 years has failed to establish any trend of degradation or aggradation in the Jamuna section of the Brahmaputra river which may be considered significant within the confidence limits set by the magnitude of errors inherent in the data. The consistently good match obtained between observed and simulated water levels over a 25 year period using the 1-D hydrodynamic model reinforces this conclusion. Specific gauge analysis computation provide further corroboration.

Therefore it is appropriate to assume that the rivers are in equilibrium and to adjust the model set up accordingly. Even if there is in fact a small underlying trend, the model results will still provide a valid quantification of the impact of any intervention or changing conditions which would be superimposed on any such trend.

3.4.4 Results and Interpretation

The 1-D morphological model has been used to predict the effect of various schemes proposed for implementation in the Jamuna River and the effect of changed boundary conditions for the river in the form of an increased sediment input to the river, for instance caused by changed land use in the catchment, and a general sea level rise due to the greenhouse effect. In addition, the 1-D morphological model has been used to investigate the sensitivity of the river response to various degrees of artificial narrowing of the river. The following scenarios have been investigated.

(a) constriction to 6000 m width

(b) constriction to 5000 m width

(c) constriction to 4000 m width

(d) construction of Jamuna Left Embankment

(e) construction of the Jamuna Multi-purpose Bridge

(f) a 50 percent increase of sediment input to the river

(g) a 0.5 m sea level rise

Each of these seven scenarios has been investigated in the following way:

- o a 100 year morphological simulation using 25 years (1966 - 1991) records of observed water level and discharge repeated 4 times as boundary condition (see section 2.3).
- o morphological simulation of the 1988 flood (close to the 1 in 100 year flood) immediately after implementation (before significant morphological changes have taken place).
- o morphological simulation of the 1988 flood using the bed levels obtained from the 100 year morphological simulation (i.e. after significant morphological changes have taken place).

In addition to the 7 scenarios described above, a 100 year baseline simulation (i.e. no changes to the system) and a 1 year baseline simulation of the 1988 flood have been carried out. Thus, in total 8 no. 100 year simulations and 15 no. 1 year simulations have been carried out. All simulations have been carried out with time steps of four hours in the hydrodynamic model and two days in the sediment transport/bed level routine.

The conclusions of the applications of the 1-D morphological model can be summarised as follows: Q ↓ What will be effect on bank erosion? Have you considered river training along with constriction by embankment

- o The time scale for river response is relatively large
- o The bed and water levels in the Jamuna are relatively insensitive to moderate constrictions of width
- o Very severe constrictions of width (say to less than 5000 m width) will give a significant increase of flood levels immediately after implementation. In the long term, when the river has adjusted to the constriction, the river bed will be significantly lower than in the existing conditions giving rise to a significant lowering of the low flow water levels. what about water depth?
- o Construction of the Brahmaputra Left Embankment will give rise to erosion of the bed in the lower reach of the river, but the effect on the water levels is modest.
- o The Jamuna Multi-purpose Bridge will only have local effects in the vicinity of the bridge on bed and water levels. Constriction scour will develop rapidly reducing the backwater effect of the Bridge. The depth of constriction scour will generally increase during rising stage and decrease during falling stage.
- o An increased sediment input to the river will give rise to an increased slope, which, especially in the upper reaches, will cause a significant increase of bed and water levels.
- o A rise of sea level in the Bay of Bengal will cause sedimentation in the river, which will migrate very slowly upstream. A General rise of water level in the Padma-Meghna confluence of 0.5 m results in the model in 0.22 m accretion in the lower Jamuna after 100 years.

reasoning

3.5 2-D Morphological Modelling

3.5.1 Introduction

After a lengthy period of data collection, processing, analysis, calibration and verification, the 2-D modelling system proved its value as a tool for developing an understanding of those aspects of processes such as channel scour development and bifurcation dynamics that present serious difficulties for physical modelling.

Calibration and verification of the model was largely carried out using the results of the three separate sets of bathymetric survey and sediment and velocity data collection undertaken in the Test Area 1 reach (see Figure 3.14) in June, July and November 1990, which provided two periods of changing conditions, relating to both rising and falling stages.

Satisfactory calibration was obtained, as illustrated in Figure 3.15, but the absence of a strong bend flow feature was recognised as a limitation on the validity of the verification. Accordingly a further test area was selected near Kazipur (see Figure 3.14) where a very pronounced bend with associated deep scour was conveniently situated for this purpose. Surveys carried out at this site in December 1990 and August 1991, during low flow and monsoon flow respectively, provided the necessary additional data. In December, float tracking was carried out and in August measurements of velocity and concentration of suspended sediment were taken together with bed samples. In the technical note "Verification of 2-D Morphological Model", (Part 9 of the Report on Model Studies) the verification is described in detail.

The bathymetry measured in August 1991 is shown in Figure 3.17. At that time the measured discharge in the anabranch was 21,000 m^3/s, the downstream water level was 15.4 m and the roughness coefficient was assumed to be unchanged from Test Area 1 with a mean depth of 7.0 m. Using these parameters, the 2-D model simulates the velocity field as depicted in Figure 3.16. Comparison of the measured velocities and simulated velocities shows a satisfactory agreement between model and prototype. The main flow amplifies near the bank in the middle of the Test Area; in the downstream portion, near the right bank, the velocity is as high as 3 m/s.

The change in bed level in Test Area 2 was simulated for 21 days with a constant discharge of 21,000 m^3/s. The new bathymetry predicted by the 2-D model is shown in Figure 3.18. When the measured changes in the prototype from December 1990 to August 1991 are compared to the simulated changes from August to September, it is seen that erosion and deposition takes place in the same regions. The absolute values of change in the bed level is of the same order of magnitude (up to 5 m) in the simulation as in the field measurements. The char in the middle is eroded at the upper end and the material is transported further down and deposited at the downstream end of the char. The main flow runs hard on the bank and causes heavy scour partly due to concentration of flow, partly due to bend scour. At the upper boundary, some differences appear. These small differences can be expected due to the natural variability of boundary conditions.

During this and other simulations it was found that using the van Rijn sediment transport model did result in some differences between the computed and observed sediment concentration values but the average level of the simulated and measured concentrations are remarkably equal (0.23 g/l). It is further known that due to the turbulence in nature, the

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Duplicate

concentration varies much more than in the model. The concentration is high at the upstream end of the shallow areas whereas in the deep channel, the concentration is less. The concentration field reveals where the river bed is under erosion.

When the sediment transport field is calculated in the model large variations in the sediment transport rates appear with spatial differences in the transport rate by a factor of up to 5. This is illustrated by the plots in Figure 3.19.

When looking downstream these fluctuations could be interpreted as "waves" of sediment with a migration speed of between 40 and 90 m/day. However the simulated migration speed of the major bed forms such as the deep scour trench is only of the order of 25 to 35 m/day.

The reason for this discrepancy between migration speed of simulated fluctuations in transport rate and migration speed of simulated major bedforms could be that in the first case only the one-dimensional case is considered whereas the latter is a direct picture of what happens in two horizontal dimensions. For instance, widening of a channel causes change in the transport rate in the direction of the current but not necessarily any change in the longitudinal profile.

In the simulation, the length of the scour hole has increased whereas the depth remained constant. The downstream movement predicted by the model was 500 m.

To assess the speed at which changes take place, it is necessary to take into consideration the fact that the flow varies significantly from December to August. In the simulation, a constant peak discharge of 21000 m^3/s was used whereas in the prototype, the discharge increased from low flow in december to high bank full flow in August. The migration in the prototype is the product of the varying flow from low to high flow conditions whereas high flow was applied in the 2-D model. The sediment transport and thus the migration speed is a function of flow velocity to a power of more than 3. Therefore, variations in sediment transport are more pronounced than variations in discharge, see Figure 3.20.

In the prototype, the length of the scour hole remained constant from December to August and the maximum depth increased slightly. The scour hole migrated 800 m downstream. Hence, based on the considerations described above, it is concluded that the simulated migration speed of the scour hole is in satisfactory agreement with prototype observation.

3.5.2 Applications and Interpretation

Full details of the design of the individual models using the calibrated system, the results and their interpretation are to be found in the BRTS Report on Model Studies. The principal features are summarised here:

<u>Bifurcation Dynamics</u>

The primary objective of this model was the assessment of the technical feasibility of influencing the development of anabranch channels by means of dredging. The results showed that whereas dredging of a relatively short pilot trench could indeed be effective in this way, the influence of the upstream approach to the bifurcation was a more important influence. A change of angle of approach of a little over 10° resulted in larger scale geometry change than that induced by the dredging. This is illustrated in Figure 3.21. The inference is that unless the approach angle can be controlled by means of training works, the effects of dredging could quickly be countered by natural planform changes. Apart from this specific conclusion, the model provided improved insight into the sensitivity of the natural switching process, observed to take place so frequently in practice, to the angle at which the thalweg approaches a large sandbar or char. The inference is that if a dual channel planform is to be stabilised the upstream approach conditions will have to be controlled within certain limits. This has far reaching implications with regard to the design of node stabilisation works in particular.

Confluence Scour

The objective of this study was to investigate the pattern of scour that develops downstream of an island or sandbar where two thalwegs meet and to compare the results of the simulation with observations and the empirical relationship presented in the JMB Study.

Velocity and depth conditions were chosen so that the model would be directly comparable with the bend simulations described below. The shape of the island was derived by studying the aerial photographs covering the areas in the vicinity of Kazipur, Sariakandi, Kamarjani and Sirajganj

The simulated value of confluence scour agreed well with data collected and analysed by the JMB study. In the situation with equal division of discharge between the two anabranches the difference between the simulated scour depth (26.7 m) and that predicted using the JMB relationship was less than 2 percent, which is well within the expected range of deviation from the mean implicit in the relationship. However the scour depth increased substantially to 34.6 m if the division of flow changed to 40:60 and the scour trench moved across adjacent to the bank (Figure 3.22). It may be concluded that in nature the scour hole will always tend to form close to one or the other bank.

Another important result of this investigation was insight into the time needed for confluence scour to fully develop. The model showed that 600 days were required before equilibrium was reached. It will therefore only be under a rare combination of conditions that the maximum value of scour indicated above will be reached. More often the seasonal changes in water level and the shifting positions of the thalwegs will interfere with the development and some intermediate depth will result. This explains why very deep scour is at present a relatively unusual phenomenon in the river; following stabilisation of the islands and channels it would become more common.

Bend Scour

The objectives of the bend analyses were:

- to determine the most sensitive parameters for river bend development as a means of quantifying the uncertainty attached to various predictors of maximum depth and velocity, whether these be formulae, physical or mathematical models;
- specifically, to verify the simple deterministic BENDFLOW programme which calculates maximum scour depth and velocity in river bends;

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- to derive relationships between h_{max}/h_{mean}, v_{max}/v_{mean} and Radius/Width that may be used for predicting maximum scour depth and velocity based on bend characteristics that are specifically applicable to the Brahmaputra conditions.

The models were based on the bathymetry observed in Test Area 2, which represents a fairly severe set of conditions with a slope of 11×10^{-5} being greater than the average for the river as a whole, and an initial bend radius of 3,000 m. The flow was taken to be the equivalent of dominant discharge giving a mean depth of 7.0 m and a mean velocity of 2.0 m/s.

The output from a numerical model is sensitive to the values assigned to a number of parameters, some of which are difficult to quantify through field observation. The degree of sensitivity was assessed by running the model with values representing the range of conditions that might be encountered in the Brahmaputra. The simulation time in all cases was 167 days real time, by which time nearly all cases had reached full equilibrium. The results in terms of both h_{max}/h_{mean} and time for development of the scour is depicted in Figure 3.23.

Comparison with the BENDFLOW model was carried out for two water depths: 4.0 m and 7.0 m. The run time in this case being 417 days. The results in Table 3.6 show that the BENDFLOW model, which is based on bed load transport only, tends to underestimate the scour while slightly overestimating velocity.

Explain – why this happen?

Table 3.6 Comparison between the 2-D model and the BENDFLOW model

Model	h_{min}	h_{max}	h_{max}/h_{mean}	v_{mean}	v_{max}	v_{max}/v_{mean}
2-D	1.46	8.09	2.02	1.53	1.94	1.26
BENDFLOW	1.79	7.15	1.79	1.51	2.01	1.33
2-D	1.47	15.07	2.17	2.03	2.53	1.25
BENDFLOW	3.12	12.51	1.79	2.00	2.61	1.31

The influence of the ratio of Radius/Width was simulated for six different cases with values ranging from 2 to 10. It was found that no scour took place for the lowest value but that in all other cases there was the expected link between scour depth and R/W (see Figure 3.24). This is consistent with both theory and observation that when a bend becomes over-tight it can no longer sustain itself. Of relevance to the understanding of bend development in the Brahmaputra is the timescale for scour: equilibrium was only reached after about 170 days of steady flow conditions, which is considerably longer than the duration of the normal high flow season.

It was found that the very deep scour observed in the Brahmaputra could not be simulated by the bend model alone, the inference being that such scour must be related to other flow conditions, such as those associated with confluences downstream of islands and flow around protruding objects, for example the situation at Fulcharighat in 1990/91. To investigate this relationship further the depth of scour was deliberately exaggerated by an adjustment to the model and the relationship shown in Figure 3.25 was derived. The very strong and significant influence of V_{max}/V_{mean} on h_{max}/h_{mean} is clearly demonstrated; an increase of the former from 1.3 to 2.1 results in an increase from 2.0 to 5.5 in the latter.

The conclusions in summary are as follows.

The study has shown that the most sensitive parameters in the development of bend scour are the slope of the water surface, grain size, bank erosion rate, eddy viscosity representing the degree of turbulence, and the variation in bed resistance over the cross section.

The last two of these are compound parameters that are used in the process description and their values are normally best determined by calibration.

The study revealed that the dynamics in bend scour development are very sensitive and a state of equilibrium is probably never reached because the parameters mentioned above change continuously in the Brahmaputra. One of the most important degrees of freedom, the horizontal movement of a bend, was fixed in the 2-D model. Instead the bank erosion was included as a lateral inflow of sediment. The dynamic interaction between bank erosion and planform movement cannot be explored with a 2-D model at the present state of development. However, the sensitivity analysis showed the significance of bank erosion on bend scour for instance if a revetment was constructed. The difference in maximum depth would be in the order of 2 m.

The 2-D model and the simpler analytical BENDFLOW model were compared and the results were found to be compatible when the sensitivity of the various parameters was taken into consideration.

The BENDFLOW model underestimates scour and marginally overestimates velocity amplification.

The modelling of the bed levels reached a state of equilibrium after approximately 170 days, when the maximum depth would be about 15 m. The results give an indication of the time scale of bend scour development (but not planform movements) although the flow would not be steady for the whole period as assumed in the model. Another approximation was that the planform geometry was fixed.

In this connection, it is important to compare the results of the JMB studies. Collected data on bend scour from surveys carried out by BIWTA gave depth below Low Water Level (LWL) ranging from 6 to 23 m. The mean value from 27 measurements was 13.4 m with a standard deviation of 3.6 m. The dominant discharge water level is almost 4 m higher than LWL indicating that the 2-D model simulated scour depths, of the order of 10 m below LWL, are less than those actually observed in the Brahmaputra.

The 2-D bend model could not simulate the development of the large scour holes at, for instance, Kazipur. However, from the verification of the model on Test Area 2, it was shown that the model was able to simulate the migration of the scour hole once it was formed. The reason why the creation of the scour hole cannot be adequately simulated is believed to be because other modes of scour, confluence and "protrusion" scour, interact with bend scour produce the critical depths. This has been studied in more detail in the model dealing with confluence scour where this explanation was confirmed.

With fixed bed levels, the maximum near bank velocity as a function of scour depth was simulated. The BRTS physical modelling showed that a ratio of upto about 2 between maximum and mean velocity is possible. The 2-D model predicted a similar ratio when the maximum scour depth was about 5.5 times the mean depth.

Flow Patterns associated with Training Structures.

The study was carried out primarily to determine whether the 2-D modelling depth-averaged modelling system could satisfactorily simulate the conditions around a groyne head or other structure that artificially distorts the flow lines. It was to be expected that in some respects the 3-D flow conditions would not be adequately represented but it was not known to what extent this would be the case.

The system was first calibrated by comparison with a physical model of a simple vertical sided groyne tested at the RRI before other configurations were investigated. Finally the proposed layout for the upgraded protection works for Sirajganj Town were simulated and compared with the physical model results.

The simulated velocity distribution around the existing Ranigram Groyne is shown in Figure 3.26. This is compared in Figure 3.27 with the simulated velocity distribution around the upstream termination of the revetment, incorporating Ranigram Groyne, which is proposed for construction under the Priority Works Contracts.

The 2-D model study and comparisons with the physical model tests of the same geometrical shape and for the same flow conditions have revealed that the 2-D model gives an accurate simulation of the flow field and associated scour only when the flow field is two-dimensional, i.e. not with cross-flows as observed near the groyne nose in the physical model.
3-D models have been developed for simulation of fully three-dimensional flow, although such models have not yet been developed to cope with sediment transport and simulation of scour and deposition. For the present, therefore, the prediction and assessment of local scour around groynes and similar structures will still have to be based on physical models such as performed for the BRTS at the RRI. The physical model has the advantage of being able to reproduce accurately the complex 3-dimensional flow field near a groyne nose.

Effect of a Local Channel Constriction

The long-term training of the river may involve the construction of cross-over (node) stabilisation at the two primary elongated cross-over reaches in the vicinity of Sariakandi and Sirajganj. The abutment protection works for the proposed Jamuna Bridge and the associated hard points of Sirajganj and Bhuapur upstream are similar in concept to such stabilisation works and may be taken as a useful example. The results of the 2-D modelling produced the following points of interest:

- immediately after construction of a 4,608 m wide constriction, and before bed adjustment has taken place, the passage of a large flood (of the order of 100 year return period) could result in a difference in water level of upto 1.0 m between the bridges and a point 4 km upstream. However once adjustment has occurred, the majority of which would take place within one season, the difference would be reduced to about half that.

- In addition to the general scouring of the river bed there will be concentrated scour in the vicinity of the training works as illustrated in Figure 3.28.

- Maximum near bank velocities of 5.0 m/s were simulated for the severe 100 year condition dropping to 3.8 m/s after bed adjustment. The flow field is illustrated graphically in Figure 3.29.

- Reducing the width of the constriction to 3,600 m resulted in an increase in the head differential of the order of 0.43 m, measured 4 km upstream of the bridge, whereas increasing the width to 5,600 m provided only about 0.14 m reduction (all values relate to nominal 100 year flood peak and bed profile adjusted to the dominant discharge).

- the significant range of water surface elevation differences that can occur across the width of the river under high flow conditions, particularly where the thalweg switches from one bank to the other (Figure 3.30). This is of relevance to the confidence which can be attributed to the flood levels simulated by a 1-D model and may also help to explain some of the variations observed in the Bahadurabad rating curve.

3.6 Conclusion

In common with other FAP studies, data limitations have proved to be the principal constraint in the use and efficacy of all forms of mathematical modelling. In many cases it is the uncertainty over datum levels that is the prime cause for concern. The time absorbed with data collection and verification has accordingly been considerably greater than anticipated and the timing of application work reflects this.

This and other subjects have been given considerable attention by FAP-25 and there is now a better understanding of the magnitude of the problem and the qualifications that must be applied to any results. The potential sources of error applicable to the BRTS situation have been identified in Annex 2 of the Second Interim Report and the consequences in relation to the accuracy of results are quantified in terms of magnitude; in most cases a more accurate quantification is not practicable.

Despite these difficulties, mathematical modelling is clearly an essential ingredient of any study of the behaviour of the river and the emphasis must be on making best use of it within the context of the recognised limitations. Future studies may benefit from this and focus their data collection accordingly.

What accuracy and extent of data is necessary to get rid of the constraint of data limitation? Is it possible & practicable to have such data? What will be the variation of result with presently available data? How perfect is the mathematical modelling technique in use for the river morphology study?

4. PHYSICAL MODELLING

4.1 Introduction

A full description of the planning, design, implementation, results and interpretation of the physical modelling programme is to be found in the separately published Report on Model Studies and the individual test bed reports prepared jointly by the RRI and BRTS. This chapter describes the objectives of the physical modelling and the scope of the testing carried out, summarises the results and conclusions and shows how these have contributed to the preparation of the master plan and the design of the priority bank stabilisation works.

The design and layout of the Priority Works structures, which include the revetment incorporating Ranigram Groyne at sirajganj, and hard-points at Sariakandi and Mathurapara, are described in chapter 9 of this report.

4.2 Objectives and Scope

4.2.1 Objectives

The two primary objectives of the physical modelling programme are: firstly, to assist with the identification of the most appropriate layout for river training works to suit the particular conditions encountered within the study reach of the Brahmaputra River; secondly, to contribute to the derivation of design values for key hydraulic parameters, such as near-bank velocity and scour depth, and to evaluate the performance of alternative arrangements for protective layers and falling aprons.

In the context of the BRTS, the principal value of physical modelling as a design tool lies in the ability to simulate complex three dimensional flow conditions, such as vortex formation, which are very important factors with regard to both scour development and the tractive forces associated with turbulent flow that act on bank protection works. The principal limitation lies in the distortions that arise from scaling effects, which means that features such as bend scour and other important major bed form dynamics cannot be accurately simulated, although some useful qualitative indication of such morphological effects can be obtained. Scaling difficulties also arise when considering features that involve the modelling of relatively large plan areas; if scale excessive distortion is to be avoided while maintaining adequate flow depth in the model for practical measurement purposes, the required plan dimensions of the model can become unmanageable.

In general, with the highly mobile bed conditions encountered in the Brahmaputra, physical modelling is accordingly seen as more useful as a detailed design aid than for applications at the planning level.

4.2.2 Scope of Physical Modelling Programme

The BRTS physical modelling programme has been undertaken by the RRI at their recently completed facilities at Faridpur under a Memorandum of Agreement with the consultant dated 11 August 1990. Model work commenced at the RRI Faridpur in December 1990, following the substantial completion of their outdoor test bed area and continued through into June 1991, although weather conditions severely interfered with the work from March onwards.

Modelling recommenced in October 1991 and was completed in May 1992. The construction of a simple but effective shelter over the test bed during this latter phase greatly improved working conditions and had a substantial impact on the period of time during which testing could be carried out.

As the overall study progressed a better appreciation of the river's characteristics has been obtained and it has become clear that the rapidity with which the morphology of the river varies means that, in contrast to less volatile rivers, site specific models have very limited value for the planning and design of river training works at a particular location; the constantly shifting pattern of the river means that such site specific testing becomes unrepresentative even before the tests have been completed. The emphasis has consequently shifted to an approach that, while being specific to the Brahmaputra river, concentrates on typical severe conditions that can occur at any point in time and at any location on the river, and on investigating the appropriate physical measures for addressing these situations. It was found appropriate however to utilise specific surveyed bathymetries, where these were suitably representative of relevant conditions, as the basis for the assessment of alternative layouts for bank protection works. This approach also provided the opportunity to compare the model results with those of the prototype.

The programme has thus taken the following form:

(a) Investigations into the relative performance of groynes and revetments as means of bank stabilisation, with the emphasis on planform. Performance was judged primarily in terms of near-bank velocity patterns and to a lesser extent on scour development; the latter using coal dust as the mobile bed medium. Three test beds, with a horizontal scale of either 1:200 or 1:300 and vertical scale 1:125, were used for this purpose, covering a range of conditions frequently encountered on the right bank of the river:

- Kazipur: representing an aggressive short radius bend causing rapid bank erosion and associated with deep near bank scour (Figure 4.1);

- Sariakandi: representing a long radius bend with moderate but sustained bank erosion and relatively mild bend scour (Figure 4.2);

- Fulchari: representing the erosion of a cusp, or headland, between two embayments by the combination of a medium radius bend with the residual effects of a short radius bend that had occupied the upstream embayment, thus causing concentrated high velocity flow at the cusp (Figure 4.3);

In all three cases the efficacy of sets of groynes was compared with bank stabilisation by revetment.

In addition the bathymetry in the vicinity of Sirajganj was modelled because of its relatively stable planform in recent years, the importance of the town protection works and the significance of Ranigram Groyne as a practical example of this type of training works (Figure 4.4).

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(b) Larger scale testing (1:50 undistorted) of alternative arrangements for slope armouring and toe protection by means of different forms of falling apron. The objectives being to determine zones requiring different sizes of armour material and to investigate the performance of the falling apron with different geometries and materials. These tests were based on a planform corresponding to the typical hard-point concept that developed from the earlier studies.

(c) Comparison of the velocity distribution and scour pattern between alternative groyne nose geometries and the correlation of these with the 2-D mathematical modelling results. These tests were also carried out at an undistorted scale of 1:100.

(d) Preliminary investigations into the use of permeable groynes and other training works that might have particular applications such as those associated with more permanent ferry ghats.

Full details of the tests carried out and descriptions of the test beds are to be found in the Report on Model Studies.

4.3 Comparison of Models and Prototype

4.3.1 Flow Velocities

Comparison of the velocity measurements taken at Fulchari, Sirajganj and Kazipur during August 1991 by the BRTS river survey team with those recorded in the models shows a satisfactory correlation. Due allowance has made for the changes in channel geometry that had taken place between the time when the original bathymetric surveys, on which the physical model test bed was based, had been carried out and the subsequent monsoon survey programme. In particular, the maximum velocities recorded in the river correspond closely to those found in the physical model and this reinforces confidence in the use of the physical model for deriving near bank velocities for design purposes. Cross-checks using 2-D mathematical modelling provided further corroboration.

4.3.2 Scour Patterns

Although it has proved difficult to reproduce the maximum depth of scour recorded in the river, the pattern of scour generally corresponds satisfactorily with observations. For example: whereas the maximum depth of scour associated with Ranigram groyne as recorded by the BRTS surveyors in March 1991 was not reproduced in the physical model, the general pattern of scour at the head of the groyne matches very satisfactorily (Figure 4.5).

The less satisfactory reproduction of the scour upstream of the groyne recorded at that time could be explained in terms of bend flow effects; these are known to be poorly reproduced in the physical model because of scaling limitations. A subsequent survey in February 1992 showed a distinct scour hole against the upstream flank of the groyne while the scour off the nose had reduced substantially. At this time the main flow was concentrated a little to the south of the groyne and angled at about 30 degrees to the general bankline; in this case the explanation could be the formation of a back eddy on the upstream of the groyne arising from the angle of approach of the flow.

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This again emphasises the importance of interpreting the physical model tests strictly in the context of the known bathymetric conditions and providing for this level of uncertainty in the design of structural works.

4.4 The Efficacy of Single and Multiple Groynes

4.4.1 Introduction

For protection or stabilisation of a reach of river bank experiencing erosion, or threatened by erosion, both groynes and bank revetments have been widely used. If groynes are the preferred solution then they are normally provided in a series and are often laid out with opposing sets on both banks in order to encourage the formation of a single navigable channel. On the Jamuna River both groynes and revetment have been tried and examples of both exist but there is no example of a set of groynes (although there are examples on the Ganges, Gorai, Teesta and other large predominantly meandering rivers). Only two groynes have survived: at Sariakandi the single Kalitola groyne was constructed in 1986/7, and upstream of Sirajganj the Ranigram groyne was completed in 1985 to provide additional upstream protection to the existing revetment stabilising the bank in the immediate vicinity of the town. These two examples provide useful test cases against which the results of the physical modelling can be compared.

The test programme can accordingly be viewed in two parts. The one focussing on these two cases of single groynes, with their rather specific objectives and their possible extension or supplementation, and the other on the scope for the construction of multiple groynes as a more general approach to the stabilisation of an eroding reach of the river.

4.4.2 Kalitola Groyne

The Sariakandi test bed was used to assess the efficacy of the existing Kalitola Groyne. Figure 4.2 shows the simulated velocity distribution during the passage of a 100 year return period flood conditions, approximating to the 1988 conditions. The limited influence of the groyne on the flow conditions can be clearly seen. The flow in the anabranch is running fairly uniformly parallel to the bankline except in the immediate vicinity of the groyne. The effect of the groyne in terms of its impact on the velocity is discernible only to about 400-500 m upstream and 300-400 m downstream of the groyne; further downstream high flow velocities occur near the bank.

The inference from this is that the length of bank that is benefiting from the protection provided by the groyne is limited to 300-400 m and this is consistent with the well defined morphological indications that may be seen on the 1:50,000 scale 1990 aerial photography. Since the effective length of the groyne is about 150 m, the length of protected bank may be expressed as about 2 to 2.5 times the effective length of the groyne in situations where the flow is approximately parallel to the bankline.

4.4.3 Ranigram Groyne

Ranigram Groyne with its large embayment upstream, temporarily in a post-accretionary quiescent phase, and relatively wide anabranch provides an example of a groyne that may experience a wide range of flow conditions. Of particular concern was the impact on the

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efficacy of the groyne, in its primary role of providing protection to the potentially failure-prone town revetment, of changes to the position and alignment of the main flow channels immediately upstream. The base case bathymetry was accordingly modified to represent four possible severe conditions (Figure 4.6):

Plan I	Three upstream flow channels
Plan II	Two principal upstream channels, one of which is partially blocked by a smaller sandbar (this is the base case bathymetry surveyed in April 1991)
Plan III	Two principal upstream channels with the smaller sandbar of Plan II removed to increase the proportion of flow in the vicinity of the groyne.
Plan IV	One consolidated upstream flow channel representing an extreme, but feasible, case in which the more easterly of the two channels has gained dominance.

In all four cases the modelled discharge was the equivalent of a 100 year return period event. The velocity fields are shown in Figure 4.6 and the longitudinal plot of near bank velocity (measured 100 m from the right bank) is shown in Figure 4.7. It can be seen that downstream of the BL school, about 2 km from the groyne, the impact of the upstream conditions is masked by the influence of the local bathymetry. In the case of Plan I the groyne is effectively deflecting the flow from the most westerly of the three channels but is having marginal, if any, effect on the flow through the other two. Plan III shows the groyne in a more effective role and its influence can be discerned down to the Jail, a distance of about 1.5 km.

In practice, following the 1991 monsoon the main flow channel shifted further east than indicated by Plan III and became more angled to the bankline, impinging on the revetment about 500 m downstream from the groyne nose and being little affected by the groyne.

In conclusion, the existing groyne at best provides some protection down to the old Jail, a distance of 1.5 km and under the most adverse conditions has little influence.

A second series of tests was carried out, using the base case bathymetry, to quantify the influence of extending the length of the groyne by 300 m and 600 m respectively. The resulting velocity distributions for a simulated 100 year return period event are shown in Figure 4.8 and the longitudinal plot similar to that of Figure 4.7 is shown in Figure 4.9.

Taking the limit of the eddy downstream of the groyne as the effective limit of influence of the groyne, it can be seen that adding 300 m to the length of the groyne only extends the zone of influence by about 300 m. With a further 300 m extension the influence is pro rata somewhat greater but still only extends as far as the BL School, thus adding about 500 m of benefitted bankline, or a ratio of 1:1.7. Further downstream, the influence of the extended groyne is shown to be negligible.

4.4.4 Efficacy of a Single Groyne

These two sets of model tests demonstrate the limited value of single groynes, particularly in a braided river such as the Brahmaputra where the approach conditions can vary so rapidly

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and widely. Under the most favourable conditions the influence of the groyne may extend up to 2.5 times its length downstream. Any change in the angle of approach or alignment of the thalweg will result in a much reduced zone of influence. At worst the formation of an embayment immediately upstream of the groyne will negate any direct upstream accretionary benefit, although the groyne will of course still function as a hard point, thereby limiting the depth of upstream embayment development, provided that its flank remains stable. On the downstream side, flow approaching at an angle of 30° to the bank, typical of an aggressive bend, will impinge at a distance of rather less than twice the length of the groyne.

4.4.5 Testing of Multiple Groynes for Bank Stabilisation

The objective was to find a suitable combination of groynes to stabilise the length of bank that was under attack in each of the typical situations represented by the three model test beds (see 4.2.2 (a) above).

In the case of Kazipur, where the short radius of the bend implied an aggressive condition with relatively high velocities and deep outer bank scour, the approach was to induce the bend to a more morphologically stable radius of about 3,000 m, while simultaneously deflecting the high velocity flow streams away from the bank. In the other two cases, involving longer radius bends, the aim was more simply confined to the latter.

The first step was to select a layout that provided the required performance, as measured in terms of the near-bank velocity distribution. In the case of Kazipur the best combination consisted of a set of five groynes varying in length from 80 to 330 m and a total length of 905 m providing protection to about 2700 m of bankline and at the same time introducing a stable embayment planform that would offer longer term advantages. The spacing between the two longest groynes was 700 m (Figure 4.10)

The Sariakandi situation approximates to providing protection on a straight bankline and the layout of 3 groynes each about 150 m long and spaced at 350 m represents a fair measure of the ratio of protected bank to length of groyne in such a morphologically mild situation. It would however be difficult to justify the construction of a groyne field on this scale in a location where erosion rates were relatively low.

It is clear that the Fulchari case is one in which groynes are not the most appropriate solution. The high velocities and deep scour are associated with a temporary confluence situation which is unlikely to be sustained for more than one or two years. The current bankline planform is not one that is morphologically desirable and therefore not one that should be stabilised at considerable expense. With the benefit of insight gained during the course of the study, situations such as this are best addressed through the construction of one or more hard-points that will allow the river some freedom of movement but limit the extent of the landward excursion of the bend. In this case a rational position for the hard-point would be on the nose of the cusp.

Following the identification of suitable layouts, the sand bed was replaced by coal dust in areas where scour was expected to occur and the tests rerun to establish the equilibrium scour pattern and the amended velocity distribution after bed adjustment had taken place. The conclusions regarding the scour patterns and the velocities are discussed in Sections 4.5 and 4.6 respectively.

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4.4.6 Efficacy of Multiple Groynes

The groyne layout selected for the Kazipur case provides an appropriate solution in a situation where it is imperative, or economically desirable, to establish a morphologically stable bankline without permitting any further loss of land. From Figure 4.11 it can be seen however that this will entail building the groynes in deep water with high velocity currents. These flow velocities will be further enhanced by the constriction of the thalweg during construction and will remain high until bed readjustment has taken place during the subsequent monsoon season. The most critical period for the survival of the groyne will thus be the season immediately following construction.

In most cases it will be necessary to design the most upstream of the groynes to remain stable following the development of a full embayment upstream, a situation which is likely to develop within ten years at most and possibly considerably more rapidly.

Thus while sets of groynes can be very effective they will tend to present a relatively costly solution that can normally only be justified in situations where the establishment of a particular bend planform is a specific objective.

4.5 Scour Depth Prediction

4.5.1 Introduction

Scour occurs naturally in the river due to the helical flow that is associated with bends and the high velocities that arise downstream of chars and sandbars where two thalwegs join together. Neither of these processes are well represented by a mobile bed physical model and use has been made of the 2-D mathematical model to quantify the effect (see Chapter 3 of this report). Conversely, the sediment transport associated with the complex flow around the nose of a structure such as a groyne cannot be fully described in 2-D terms and the use of physical modelling can, with certain provisos, provide a better indication of the form and depth of scour that can be expected to develop (see Section 4.3.2).

Scour associated with groynes has been measured both in the small scale test beds, using coal dust as the mobile medium, and in the larger scale outdoor flume tests where velocities were sufficiently high for the sand bed to become mobile over the full width of the bed. The designs of the tests and the results are described fully in the physical modelling volume of the BRTS Report on Model Studies.

4.5.2 Scour Pattern Associated with Multiple Groynes

The Kazipur Test Bed was used for the principal set of tests aimed at investigating the pattern of scour that may be expected to develop after stabilisation of a typical anabranch bend by means of a series of groynes. An appropriate layout of the groynes was first established using a fixed sand bed and then the sand was replaced by coal dust in those areas where scour or accretion and was expected. The model was then rerun until equilibrium was established.

The results are shown in Figures 4.10 and 4.12. It can be seen that the pattern of scour and deposition is quite regular with the deepest scour occurring off and to the downstream of each groyne, where the effect of the concentration of streamlines is superimposed on the effects of the formation of separation vortices. As shown in Table 4.1, the depth of scour does however vary considerably, whether measured relative to the original bed level or with reference to the water surface.

Table 4.1 Maximum Equilibrium Scour Depths measured at Groyne Noses in the Kazipur Model.

Groyne (m PWD)	Original BedLevel (m)	Water Depth 'h'	Maximum Scour s'	Water Depth After Scour h_{scour} (m)	Scoured BedLevel (m PWD)	Scour,s Depth h
B	-1.0	18.25	3.8	22.05	- 4.8	0.21
C	2.0	15.25	8.0	23.25	- 6.0	0.52
D	6.2	11.05	9.2	20.25	- 3.0	0.83
E	6.3	10.95	13.3	24.25	- 7.0	1.21
F	-6.0	23.25	6.0	29.25	-12.0	0.26

Note : WL = + 17.25 m PWD, Anabranch Discharge, Q = 17,000 m^3/s,

It is seen that the maximum depth of scour was 13.3 m at Groyne E, corresponding to a ratio of 1.21. The deepest scour level was reached at Groyne F where the scour reached level -12.0 m PWD and the total water depth was 29.25 m. It is interesting to notice that the deepest scour hole occurred where the water depth prior to scour was about 11.0 m (groyne D & E) while the deepest scour level was reached at Groyne F which prior to testing was in very deep water, (h = 23.25 m).

The maximum depth of scour relative to water level (29 m) occurred downstream of the final groyne in the series where the original bed level was greatest. This is of the same order as measured maximum scour depths in the river associated with bends and confluences and probably does not represent the worst combination of hydraulic and morphological conditions that could develop. The tests with single groynes at a scale of 1:100 (non-distorted) have shown scour depths exceeding 30 m (see the Report on Model Studies, Part 4). The displacement of the thalweg is clearly shown in Figure 4.11.

4.5.3 Scour at the Nose of a Single Groyne or Hard-Point

Of primary concern for the detailed design of works for early implementation is the pattern of scour that can be expected to occur off the nose of a groyne or the upstream termination of a hard-point. The latter may approximate to the conditions at the nose of the groyne after several seasons have passed and an embayment has developed upstream. The main difference between groyne and hard-point nose in this respect, other than geometry, is that the latter will not experience the accentuated flow velocities associated with the realignment of the thalweg during the first season.

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Scour of the nose of a single groyne was studied both in the distorted live bed model for Sirajganj and in non-distorted models of larger scale. Figure 4.13 from the model study on Sirajganj bathymetry gives an impression of the size of such a model (top) and of the deep scour developing off the toe of the structure (bottom).

A series of tests were carried out in the outdoor flume at the RRI. The majority of these were at an undistorted scale of 1:100. The tests were performed with a horizontal bed and a water depth of 20 m at test initiation.

A number of different structures were tested including a BRTS-type "hardpoint" revetment, groynes with various geometrical characteristics and permeable groynes in the form of structures consisting of spaced cylindrical piles.

The test conditions imposed approximated to the most severe conditions that can be expected to occur at a structure built on the right bank of the Brahmaputra River. The tests showed the following maximum water depths near the structures after equilibrium scour had occurred.

BRTS-type revetment

The scour at the revetment is concentrated near the upstream termination, and the maximum scour depth reached 29 m relative to the 100 year flood water level.

Groynes

Different groyne geometries and side-slopes were tested. The maximum water depths found ranged from 29 m to 40 m. The largest scoured depth of 40 m was found at the upstream groyne in test 17, the model study of a ferry ghat layout.

4.5.4 Timescale of Scour Development

The timescale of scour development is primarily of importance in assessing the speed with which the river will adjust to the new imposed regime and thus the duration of the period in which the higher velocity concentration at the nose can be expected to last.

Figure 4.14 shows the speed of scour development for two of the tests carried out using the Kazipur test bed.

The process of scour depth development as function of time is seen to take the form of a "decay" curve where the rate is largest in the beginning and where at the end the scour depth asymptotically approaches the equilibrium situation. The conversion of the rate of scour from model to prototype occurs according to complex rules and no definite conversion factor can be established. However it is known from sediment transport calculations and practical experience that deep scour can develop in a matter of a few weeks to a few months which means that the scour can take place within one high flow season. Ranigram Groyne was constructed in 1985 and already one year later, in August 1986, the scour hole was fully developed.

This also means that the river bed in the vicinity of a major structure such as for example Ranigram Groyne is under continuous development in an attempt to adjust itself to the changing water level and flow velocity over the season. This is a complex relationship as the flow conditions and water level rarely remain constant for more than a few days.

4.5.5 Parameters Influencing Scour Depth

The river training structures to be built in the Brahmaputra as part of the BRTS programe for priority works and the long term master plan for stabilisation of the BRE will all have to rely on an assessment of the maximum water depth likely to occur within the lifetime of the structure.

The maximum depth, i.e. the lowest bed level that is likely to occur, is dependent on the following factors.

(a) normal variations in the channel cross-section geometry that occur in the course of a normal hydrological year, irrespective of any change in the channel plan geometry;

(b) variations in the channel plan geometry such as the development of an aggressive bend, char approaching the bank, or anabranch confluence characteristics

(c) passage of bed forms (dunes)

(d) local scour produced by the structure itself due to its distortion of the flow field.

Data that is available for this purpose is derived primarily from BWDB surveys, the BRTS surveys and the Jamuna Bridge Studies (JMBS). With respect to local scour produced by the structures, data from literature and the physical and mathematical studies performed by BRTS was included in the analysis.

The physical model testing undertaken to investigate the influence of these parameters on the development and magnitude of scour is described in Appendix B. Of immediate relevance to the design of the phase 1 structure are the following conclusions concerning scour depth:

Maximum water depth for revetment design (straight sections)	= 29.0m
Maximum water depth for upstream terminations and noses of groynes	= 33.0m

4.6 Flow Velocity Prediction

4.6.1 Introduction

The design of the armour layer forming the principal protection to the bankslope or groyne core material is very sensitive to the velocity that can be expected to occur during the structure's design life. Since it would be uneconomical to design for the worst case, it is necessary to establish the relationship between velocity and probability of exceedance so that an appropriate risk level may be selected.

The derivation of the appropriate design velocities for structural works on the right bank of the Brahmaputra is described in Appendix C. The procedure involves three steps: firstly, establishing the variation of the mean velocity due to major bedform dynamics, secondly the imposed effect of natural planform features such as bends and confluences and thirdly the further influence of artificial features such as revetments and groynes.

It is these second and third steps where physical modelling has an important contribution to make. By measuring the range of velocity amplifications found in the various models it has been possible to establish relationships applicable to different planforms and types of structure that are specifically applicable to the Brahmaputra conditions.

4.6.2 Velocity Amplification in River Bends

The model studies carried out using the Fulchari, Sariakandi, Kazipur and Sirajganj test beds all involved the study of the velocity distribution in the vicinity of bank parallel revetments or in river bends. The studies were for two of the sites performed with different configurations of the upstream bathymetry including varying the proportions of major sandbars and the distribution of flow between secondary anabranches.

Table 4.2 shows the results of the model studies with respect to mean velocity (Vbar) and maximum velocity (Vmax) as well as the ratio, Vmax/Vbar. This presentation has been made in order to compare the different situations and thereby derive general conclusion that can be applied in the whole study area. It is apparent that although the mean velocity and the maximum velocity differ from one model to another, the ratio Vmax/Vbar does not show as much variation and that there is a definite correlation between the curvature of the bend and this amplification factor.

The lowest amplification of 1.50 was found in the Sariakandi Model bed in which the bend has a relatively long radius. For the other three model beds the amplification ranged from 1.81 to 2.15. The largest amplification was found in the Kazipur Model bed in the near bank section of a concave bend which was close to the most severe condition found on the right bank.

Table 4.2 Maximum Near Bank Velocity in unprotected Bend

Site	Q (100-yr) in Anabranch (m^3/s)	Cross Section Area of Anabranch m^2	Average Velocity V(m/s)	Maximum Measured Velocity V_{max}(m/s)	Velocity Amplification Factor Vmax/V
Fulchari[1/]	29,000	13,300	2.2	4.2	1.91
Sariakandi[2/]	13,500	9,500	1.4	2.1	1.50
Kazipur[3/]	15,500	15,500	1.3	2.8	2.15
Sirajganj[4/]	25,000	15,625	1.6	2.9	1.81

Notes:

1. Maximum velocity in section 8 off the railway ferry ghat.
2. Maximum velocity in section CS#10.
3. Maximum velocity in section CS#11 south of bend off the hospital/college
4. Maximum velocity in section C 52 north of the steamer ghat

4.6.3 Velocity Amplification Associated with Multiple Groynes

Series of tests were carried out using the Sariakandi and Fulchari test beds to investigate the velocity field around a set of multiple groynes.

Tests 2 and 4 from the Sariakandi test bed, representing the conditions before and after the introduction of groynes, provide a good comparison of the respective velocity distributions. In Table 4.3 the measured velocities at cross-sections 7 and 9, corresponding to two groyne locations, are compared for the simulated 100 year return period discharge.

Table 4.3 Maximum Velocity Amplification Near Groyne Head. (Sariakandi Model Bed)

Profile Ref	Distance From Right Bank (m)	Velocities [m/s]		Amplification
		No Groyne: Test 2	With Groyne: Test 4	
7	323 431	1.60 1.55	2.25 2.20	1.41 1.42
8	206 310 412	1.95 1.60 1.55	2.40 2.50 2.25	1.23 1.56 1.45

The amplification is seen to be in the range 1.23 to 1.55. This upper value is slightly higher than might be expected from idealised flume experiments but this can be attributed to the fact that these measurements relate to a fixed bed model, which represents the situation immediately after construction and before bed adjustment has taken place.

A similar comparison of the results from tests carried out in the Fulchari Test Bed are given in Table 4.4.

Table 4.4 Velocity Amplification Near Groyne Head (Fulchari Model Bed)

Profile Ref	Distance From Right Bank (m)	Velocities (m/s)		Amplification
		No Groyne: Test 2	With Groyne Test 4	
CS#7	192 288 432 576	3.83 3.46 2.83 3.27	3.77 4.21 3.83 4.21	0.98 1.22 1.35 1.29
CS#8	126 252 378 566	3.33 4.33 2.71 3.39	- 5.55 4.71 4.77	- 1.28 1.74 1.41
CS#9	126 252 378 568	3.70 3.96 2.52 2.33	4.58 5.91 4.14 3.27	1.23 1.49 1.64 1.40

In this case the velocity amplification was found to be in the range 1.35 to 1.74, which is higher than the Sariakandi example.

The results exhibit scatter, as whould be expected given the complexity of the morphological conditions, but it can be concluded that for the conditions prevailing in the Brahmaputra, a groyne may be expected to amplify the flow velocity in the vicinity of its nose by a factor of 1.5 ± 10 percent but that in some circumstances this factor can be as high as 1.75. As noted earlier, the tests were performed for fixed bed and therefore represent conditions immediately after construction. Further tests with coal dust introduced to allow scour to be simulated resulted in a reduction of the maximum velocities by about 10 percent. Therefore after bed adjustment has taken place, which may take one to two seasons, the velocity amplification at the nose of a groyne could be expected to be in the range 1.4 ± 10 percent. Superimposed on this, however, will be other effects associated with the migration of major sandbars and, as discussed in Appendix C, the factor to be applied to the mean velocity in the anabranch must also take these into consideration.

4.6.4 Velocity Amplification associated with Hard-Point Terminations

Hard-points when constructed will typically be set into the current bankline and therefore will initially result in very little interference with the river flow pattern. With the passage of time, however, it is to be anticipated that an embayment will develop upstream of the structure resulting at worst in a situation that in terms of approach conditions is not unlike that at the nose of an isolated groyne.

In order to make provision for this development, it may be necessary to increase the size of the armour material over that part of the upstream termination that may become exposed to the higher velocity flows that theoretically could arise. Such heavier armour is both costly in itself and will be more difficult to place without the use of heavy machinery and plant; it is clearly desirable to minimise its use.

In view of this concern, a series of tests at an undistorted scale of 1:50 were carried out in the outdoor flume at the RRI. The primary aim was to investigate the performance of the falling apron, but this provided the opportunity concurrently to study the pattern of velocity distribution for a range of flow conditions and to determine which parts of the structure were most susceptible in this way. This larger scale made it possible to see clearly the vertical distribution of velocity that could not be seen in smaller scale test beds. The full details concerning the design and implementation of the tests is given in part 4 of the BRTS Report on Model Studies.

It was found that the effect of simulating the development of an embayment alone was to increase the maximum velocity close to the revetment by a factor of between 1.10 and 1.20, depending on the setting depth of the falling apron. This represented a situation in which the embayment had developed but flow was approaching at a shallow angle, as would typically occur when the embayment was in the process of being abandoned and a longer radius bend was attacking the headland on the downstream end of the embayment (e.g Fulchari in the monsoon season of 1991).

Figure 4.15 shows the velocity distribution at four sections in the model for Test 1C which is without an upstream embayment and Figure 4.16 shows the equivalent for Test 2C in which the embayment has been simulated. For reasons explained in the Report on Model Studies, these simulated velocities are higher than those that would normally actually occur in the river (it is estimated that a maximum depth averaged near-bank velocity of 4.0 m/s has a less than two percent probability of occurrence somewhere in the river during the passage of a 100 year return period flood) but the relative values and the distribution are valid. From inspection of the results it may be inferred that the maximum near-bank velocity that is likely to occur in the vicinity of the revetment is of the order of 1.3 times the equivalent velocity in the absence of the structure. It is to be emphasised that this relates to a situation where the full extent of the upstream termination has become exposed.

The influence of the shape of the upstream termination, or the head of a groyne, on the velocities in the immediate vicinity were also studied in the course of tests carried out on a specially built model for Sirajganj. The primary objective in this case was to select the most appropriate shape for the junction between the existing Ranigram Groyne and the proposed new revetment (Figure 4.17). Tests were carried out with both the 2-D mathematical and physical models and there was a good agreement between these two approaches with regard to the velocity measurements (see also Section 3.5.2 of this report). It was found that the amplification factor for the existing groyne was about 1.4 and that the new arrangement with a 100 m long hammerhead gave almost identical maximum velocities. If the hammerhead length was increased to 200 m the velocities along the upstream shank of the groyne were reduced marginally but the maximum velocity along the revetment face increased by almost 10 percent. The 100 m long hammerhead was found to substantially decrease the velocity along the shank and this layout was consequently selected.

4.6.5 Interpretation of Results

seems low

The general guideline for estimation of the maximum velocity developed around the nose of a groyne is to multiply the mean velocity upstream by 1.4. This is a general "rule-of-the thumb" based on laboratory flume studies and other prototype observations and presumes that the bed conditions are uniform and that the flow upstream is approximately parallel to the bank. Conditions in Brahmaputra diverge substantially from these assumptions and the ever-changing bathymetry means that a diverse range of approach conditions can be experienced.

Nonetheless, the results of the several model tests that have been carried out all confirm that this nominal figure is the correct order of magnitude, although in the real world of the Brahmaputra river it is difficult to define what is meant by mean upstream velocity. For this reason It is considered to be more appropriate to relate to maximum near-bank velocities, which have some physical meaning and can be measured.

On this basis, and using the multipliers derived from the physical and mathematical model studies, it is concluded that the exceedance probabilities for maximum velocities in the vicinity of a structure may be as in Table 4.5 and that after taking into consideration the 30 year design life of the structure and adjusting for probability of occurrence at a location on the right bank, appropriate design values for the velocities to be used for the sizing of armour layers would be as shown in Table 4.6. Attention is drawn to the fact that these are based on the conservative criterion of an acceptable risk of failure of only 1 percent.

Table 4.5 Maximum Velocities (m/s) for given Exceedance Probability for a 100 year return Period

Parameter	Exceedance Probability (%)					
	2	5	8	10	20	50
U-mean (m/s)	2.3	2.2	2.1	2.0	1.8	1.5
U-max (m/s)						
no structure	4.0	3.6	3.3	3.2	2.8	2.2
Groyne (1.4)	5.7	5.1	4.8	4.6	4.0	3.0
Revetment (1.1)	4.5	4.0	3.7	3.6	3.1	2.3
Revetment (1.2)	4.8	4.3	4.1	3.9	3.4	2.6
Revetment (1.3)	5.2	4.7	4.4	4.2	3.7	2.8

Note: The figures in brackets denote the mean amplification factor applied.

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Table 4.6 Design Velocities for Different Structural Components

Type of Structure	Amplification Factor	Design Velocity (m/s)
Revetment Straight Section	1.1	3.7
Revetment Upstream Termination	1.3	4.4
Head of Groyne	1.4	4.8

Note: Based on a 1 percent probability of exceedance of design conditions in the project life (30 years).

4.7 Revetment and Falling Apron Design

4.7.1 Introduction

The use of a falling apron as a means of providing protection in the event of scour at the toe of revetment used as bank protection, or at the nose and flanks of a groyne, has a long history in the Indian sub-continent and elsewhere in the world. There has, however been very little systematic investigation of the performance of the falling apron in general and none that has been published regarding the conditions that are encountered in the Brahmaputra. The relatively high cost of armour material in Bangladesh together with the rather poor historic performance of slope armouring along the right bank of the Brahmaputra raises a number of issues that can appropriately be addressed through the use of large scale physical modelling. These issues include the selection of the material and sizing of the armour, the geometry and setting depth of the falling apron and the use of filter layers.

The master plan study has led to the conclusion that the most economical means of stabilising the right bank of the river is to construct a series of hard-points, consisting of bank revetments with upstream and downstream terminations to accommodate the formation of embayments and cross-bars to prevent outflanking of the structure.

The falling apron consists of armour material dumped at the toe of the structure. This material is sacrificial in the sense that it is allowed to move to form a protection of the lower part of the slope of the river bank should scour/erosion occur below the existing river bed on which the structure is built. On the Brahmaputra the scour can in many places be as great as 16-18 m below the original bed level, and consequently the size of the apron becomes substantial. Although there are well established standard design approaches (e.g. Indian Practice), these are all developed for rivers with distinctly different characteristics. No formulae or other rational means of determining an appropriate design for the Brahmaputra exists; for which reason the programme for revetment tests was formulated.

The key aspects that are addressed are:

(a) Velocity distribution, leading to block size determination

(b) Scour pattern and falling apron response to scour and velocity leading to apron geometry and setting depth criteria.

4.7.2 Model Characteristics

The scale of the undistorted model used for this series of tests was 1:50, from which the following scales are derived by use of Froude's Model Law.

Length Scale		= 1:50
Flow Time Scale	$1:50^{½}$	= 1:7.07
Velocity Scale	$1:50^{½}$	= 1:7.07
Volume Scale	$1:50^3$	= 1:125,000
Discharge Scale*	$1:50^{5/2}$	= 1:17,678

* Discharge scale is equal to volume scale divided by time scale.

The blocks in the model were made from cement mortar having a density of 1.96 g/cm^3, which is very close to the expected density of the concrete in the prototype, if khoa (crushed brick) is used as aggregate. For simulation of concrete blocks in a Froudian Model the size of the blocks should be scaled with the length scale which in this case is 1:50. Therefore a 60 cm block is scaled to be 12 mm in the model. The water in nature and in the model is fresh water. Therefore, the density of the block should be the same in model and prototype to fulfil the Froude Model criteria.

Each model was run with the average velocity being increased through a series of steps (stationary conditions in terms of flow discharge). The duration of flow for each step was selected on the basis of a calculation of the sediment transport in the model in order to arrive at equilibrium scour conditions in each test sequence. The duration of each step was approximately 2½ hours.

The model constituents were:

- Sand of approximately d_{50} = 0.2 mm

- One layer of very thin cotton fabric, representing the geotextile fabric;

- Concrete blocks in the form of cubes with density of 1.96 g/cm^3

- In some sections (dummy sections) where measurements were not carried out the sand was covered by khoa, of approximate size range 10 to 20 mm.

Typical model layouts and sections are shown in Figures 4.18 and 4.19. It can be seen that a deep channel was constructed in front of the apron at the approximate expected scour depth of the apron. By this means it was possible to ensure that the scour took place at the toe of the apron and that accordingly the apron would commence launching as soon as the sediment transport became active on the slope below the apron.

These tests were designed to be representative of the more severe conditions that are encountered along the right bank of the river. The flow conditions simulated ranged from normal flood flow conditions to 100 year return period events and beyond in order to explore the full range of possibilities and study the modes of failure. The velocity measurements reported here and in the detailed reports must be interpreted accordingly.

4.7.3 The Model Test Series

The series consisted of 12 models, each addressing a specific issue. The majority (those with a reference S1) were based on of the situation at Sariakandi and Kazipur in 1990/91. At that time the deepest bed level was about 20 m below the 100 year return period flood level and the apron was set at this bed level. The deepest design scour depth had been assessed at 29 m below the 100 year water level, and consequently the apron to be tested was designed to allow for a scour depth of 9m below the apron setting level. The apron geometry was that derived from the preliminary phase of the study (see the First Interim Report for details). A typical plan and cross-sections of this model layout are shown in Figures 4.18 and 4.19. Other variations to the setting depth and apron geometry were also tested, as shown in Table 4.7.

Full details of the models, the observations and interpretation of the results that are summarised below are given in the physical modelling volume of the BRTS Report on Model Studies.

<u>Model I</u>

The runs using the first model layout resulted in a considerable amount of sand being deposited on the revetment face, indicating that high velocities were not developing in this vicinity. This situation is representative of conditions shortly after completion of construction but does not provide information on the more severe conditions that can develop with time. (See Figure 4.20, upper photograph).

<u>Model II</u>

For the second model the "flood plain" was lowered to simulate the development of an embayment upstream of the structure. This resulted in considerably higher velocities along the revetment face, to the extent that displacement of a large number of blocks occurred in both the slope armouring and the apron (see Figure 4.20, lower photograph). This failure was associated with a maximum velocity in the vicinity of the revetment face of 5.2 m/s, the location of which coincided with the centre of the damage.

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Table 4.7 Schedule of Revetment Tests - Revised Programme of 8 December 1991

Model	Tests	Configuration	Features	Objective
I	A,B,C,D	Sariakandi/ Kazipur Schematised	Single structure scour depth 9m U/S. bathymetry-high. Apron width 18.5m. Level -9.5 m PWD.	To study the stability of a typical revetment and mode of launching of apron and flow field near the structure.
II	C	Sariakandi/ Kazipur Schematised	Same as Model I except that U/S. bathymetry - Low.	Same as Model I but with deeper water upstream to simulate the effect of an upstream embayment
III	B,B,B_2,C	Sariakandi/ Kazipur Schematised	Same as Model II	To test initiation of damage for 60 cm blocks
IV	C,C_1,C_2	Sariakandi/ Kazipur Schematised	Same as Model II but 75 cm blocks were placed in the front half of the revetment and apron.	To test initiation of damage for 75 cm blocks.
V	B,C,C_2	Mathurapara Schematised	Single structure, Scour depth 16.64m, Apron width 30.0m, Apron thickness 3 m level + 7.0 m PWD.	To study the behaviour of a revetment for shallow water applications and deepest scour.
VI	B,C,D	Mathurapara Schematised	Single structure Scour depth 16.64m, Apron width 15.0m, Apron thickness 6m level + 8.10m PWD.	Same as Model V but with an apron of half width but double thickness and to study the feasibility of placing geotextile under apron.
VII	B,B_1,B_2,C	Sariakandi/ Kazipur Schematised	Same as Model II but construction was performed under flowing water upto LWL.	To test the stability of revetment and apron constructed by random dumping of blocks upto LWL.

Table 4.7 (Continued) Schedule of Revetment Tests - Revised Programme of 8 December 1991

Model	Tests	Configuration	Feature	Objective
VIII	B,B_1,B_2	Sariakandi/ Kazipur Schematised	Same as Model II but 40 cm blocks were placed at the critical zone of the revetment slope.	To determine the critical velocity for the initiation of damage to 40 cm blocks.
IX	B,C,D	Fulchari ghat Schematised	Two structures, high bathymetry between the two structures, boulders as apron material for down-stream structure.	To study the bank stabilization by using two structures and to study the feasibly of using boulders as apron material,
X	B,B_2,C,C1, D	Sariakandi/ Kazipur Schematised	Same as Model II, but revetment slope with one layer of blocks.	To test the stability of a revetment constructed with one layer of blocks on the slope.
XI	B,C,D	Sariakandi/ Kazipur (U/S. structure) and Mathurapara (D/S. Struc.) Schematised	Two structures with deep water between them. Apron width 30.0m and 29.0 m, Apron level +5.2m and +10.0m PWD respectively. Quarry stones were placed below apron of U/S. structure.	To test the behaviour of an apron above graded filter and to test the stability of an apron with geotextile underneath.
XII	B,C,D	Sariakandi/ Kazipur Schematised	Same as Model II but with the whole structure located in the bend section of the flume. The U/S. bathymetry was low.	To test the stability of a typical revetment constructed in the bend of a river.

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Model III

Since it was not clear whether the failure of the slope armour was a consequence of the apron failure or an independent process, the third model run was made with the same physical parameter values as the second but with smaller flow increments so that initiation of failure could be diagnosed. A very similar pattern of velocity and scour was observed to that of Model II. Displacement of blocks, in the same locality, was initiated when the measured maximum point velocity was between 4.9 and 5.0 m/s. The apron suffered less damage than in Model II, showing that the revetment slope failure was linked directly to the high local velocities along the face and not a consequence of apron failure.

Model IV

For the fourth model run the simulated 60 cm blocks were replaced by 75 cm blocks but features were unchanged. The flow was increased to the maximum capacity of the facilities without any damage resulting to the revetment and at the point of maximum scour less than 60 percent of the apron volume had been launched (Figure 4.21)

Model V

In some situations it will be necessary or desirable to construct the revetment when bed scour is not active and the bed level is almost exposed at low water level. The options in such cases are to place the apron at the existing bed level or to dredge a trench to a predetermined depth in which the apron is then placed. The former has obvious practical and cost advantages while the latter minimises the reliance on the apron performing as intended and uncertainty attaching to the absence of a filter layer below the launched armouring. The latter also has geotechnical ramifications because the launched slope of 1V:2H is considerably steeper than the 1V:3.5H of the formed slope above the apron and the overall stability factor of safety of the revetment will be reduced. When selecting the setting depth of the apron there has to be a trade-off between these factors and it is necessary to know how the apron performance may vary with setting depth.

The bathymetry surveyed at Mathurapara in August and September 1990 and the preliminary revetment apron design prepared in August 1991 was used as the basis for this model.

The bed profile development as the discharge was increased is shown in Figure 4.22 and the velocity distribution is shown in Figure 4.23. The apron was observed to launch in a very even manner, suggesting that from this viewpoint the higher setting depth did not present any problem. Only about 50 percent of the apron had launched by the end of the test run with maximum discharge leaving a berm which helped to maintain the overall slope of the section and thus its stability.

Model VI

The apron geometry used in previous models was based closely on Indian practice but there are on occasions constructional advantages in using a shorter and thicker apron with the material more concentrated at the toe of the revetment slope. This section is commonly used by the US Corps of Engineers for larger river bank revetments (e.g. on the Mississippi River).

This model was designed to study how such an arrangement would perform under conditions otherwise similar to Model V.

As an additional test, the cotton fabric was continued under one portion of the apron length only while the remainder was left as a control.

It was found that this apron launched satisfactorily over the section without the underlying fabric but where the fabric was present the blocks tended to slide down the slope leaving the fabric exposed. In the prototype this would lead to severe flapping of the fabric and its rapid destruction. It was also observed that maximum near bank velocities were marginally but significantly lower than for the previous model run.

The main disadvantage of this arrangement is that after launching the remainder of the apron offers little in the way of a berm; instead there is a concentration of mass at the top of the launched slope, which is an adverse feature from the viewpoint of slope stability. This together with the fact that the deep scour is closer to the bank has a significant influence on the factor of safety against rotational slip failure.

Model VII

The layout of this model was identical to that of Model II except that the blocks below low water level were placed by dropping from the simulated water level, as they would be in the prototype, instead of being hand placed in the dry.

It was found that the launching pattern of the apron was unaffected and the revetment armour behaved equally satisfactorily. A small reduction in maximum velocity was observed which may be attributed to the greater roughness offered by the more uneven exposed face of the block layers.

Model VIII

In this case the same model conditions were repeated except for the use of simulated 40 cm blocks for both revetment and apron instead of 60 cm blocks. The objective being to determine the threshold velocity for displacement of this size of block, and thereby have more data to establish a functional relationship between velocity and initiation of block movement.

It was found that movement of the blocks was initiated for a maximum near slope velocity of about 4.1 m/s. In all other respects the performance of the revetment and apron was similar to earlier tests.

Model IX

In this model two "hard-point" structures were introduced in to the flume (Figure 4.24) to study how the upstream structure influenced conditions at the downstream site and to provide increased opportunity for trying out alternative armour materials and configurations.

The simulated conditions at the two locations were similar in relation to the pattern of velocity distribution but with some greater concentration of higher velocities near the revetment in the downstream location and about 10 percent higher maximum velocity. The difference can be due to a number of factors. One is that flume bed was horizontal at the start of the test while the water is sloping (gradient approx $5x10^{-4}$). The distance between the models was about 14 m and the water level difference thus about 7 mm or 2-3 percent of the depth.

The revetment of the upstream structure was composed of single sized 60 cm blocks and its apron consisted of a mixture of 40, 60 and 75 cm blocks, in equal proportion by number. The downstream structure had a revetment consisting of 60 cm blocks in the upstream half and and 75 cm blocks in the downstream half. The main object of the downstream structures was to test on apron formed from small rounded pebbles representing river worn boulders in the 50 to 100 cm range.

Following the completion of the high discharge run, with recorded maximum near revetment velocity of 5.2 m/s and 5.3 m/s at upstream and downstream revetments respectively, it was found that the upstream revetment of 60 cm blocks had failed, as would be expected at this velocity, the pattern being very similar to that of Model II. The apron of mixed blocks was also completely depleted over one section, indicating that there had been substantial lateral displacement. It may be concluded that the mixture of 40, 60 and 75 cm blocks performed no better than one of single sized 60 cm blocks (Model II).

There was no failure of the downstream revetment or apron despite the velocity of 5.3 m/s. This indicates the scatter in armour stability of the type of structure subjected to testing and the necessity for adopting a conservative design curve (functional relationship between velocity and initiation of movement).

The stone apron of the downstream structure launched in a manner that was indistinguishable from the cubic blocks of earlier tests. Less than 50 percent of the volume was launched but this can be attributed to the lesser depth of the slope below the apron that was exposed to scour compared with earlier tests (see photograph in Figure 4.25). In conclusion single sized cubic blocks or boulders are considered equally suitable as construction material for the launching apron. The launched apron of boulders is shown in the upper photograph of Figure 4.28.

Model X

The general layout of this Model was identical to that of Model II except that a single layer of 60 cm blocks was hand placed on the slope. The opportunity was also taken to observe more specifically the direction of movement of the blocks in the apron by painting some blocks to act as tracers.

Despite velocities being increased to 5.2 m/s, i.e above the level that resulted in failure in other models (V=4.9 m/s), the revetment remained intact and the apron launched in the usual uniform manner. It is important here to mention that the blocks were well placed to minimize the cavities in between and that they were placed directly on the geotextile. It is further important to mention that one should expect slightly lower velocities on the surface of a one layer system of hand placed blocks than at a two layer system of randomly placed blocks that will allow some flow inside the two layers. This may contribute to the explaination of the behaviour of the two situations tested. Another factor is that the single layer provides a smoother surface and therefore creates less turbulence.

It is notable that the scour depth measured at Profiles III and IV was very similar to that observed in earlier comparable tests but that the apron had been almost fully launched by the completion of the final full discharge run, which resulted in a maximum velocity of 5.2 m/s. The face covered by the launched apron material had assumed a slope which was only marginally steeper than the preformed revetment slope above.

It may be concluded that a single layer of 60 cm blocks will perform as well as a double layer and, if the results of this test are to be taken as representative, may actually out-perform the latter. However under field conditions it would be very difficult to reproduce this standard of finish. The main inference is that there would appear to be no advantage to be gained from providing additional thickness of the revetment armouring. Provided that there is sufficient material to ensure complete single layer coverage then this should suffice. The results of this test series were used in the final design of the structures where one layer of well placed blocks is used above Low Water Level (LWL) + 2.0 m, where block placing can be performed accurately under strict control with subsequent monitoring (in the dry season).

Model XI

Two structures were built to the same overall layout of Model IX but in this case the apron design for the upstream structure contained some experimental features arising from the results of the earlier tests. The 20 m wide apron consisted of between three and four layers of 60 cm blocks placed randomly over a khoa layer representing a coarse crushed rock or khoa layer in the prototype with size range 5 to 35 cm. In addition a 10 m wide horizontal berm was introduced between the apron and the toe of the revetment and this was protected with three layers of blocks. A fabric was placed under the revetment and berm but not under the apron.

The downstream structure had an apron 29 m wide and 5 m thick. In the upstream half the apron blocks were of 60 cm size and in the downstream half 75 cm size. In the downstream half the geotextile was laid in under the apron. A 10 m wide unprotected berm of sand was left between the apron and the preformed deep channel.

In both cases the apron was placed at a level corresponding to the high river bed found at places with very shallow water like Mathurapara instead of being inset into it as was the case in all earlier tests. In other words the test was aiming at studying whether the structures could be built without dredging at the toe prior to apron placing.

Once again the velocities at the downstream structure were generally higher than those at the upstream structure. At the intermediate discharge (3.2 m/s target velocity in the prototype) the composite apron of the upstream structure launched in a very uniform manner with an evenly distributed mixture of the blocks and the underlying crushed rock material. However with the higher discharge (3.5 m/s target velocity and maximum velocities of about 5.0 m/s) the apron and the berm behind it were severely damaged and the simulated crushed rock become widely dispersed. This material did not appear to have any beneficial effect and there is some evidence that it may result in larger gaps between the launched blocks, which in turn would increase drag forces and induce earlier movement as velocities increase.

The downstream structure performed better all round, although some lateral block displacement did take place at the full discharge. Once again, the portion of the apron that

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was laid on fabric displayed a very much less even distribution of the armour blocks, with substantial areas of the fabric being exposed as the blocks slid down the slope.

Model XII

The objective of this final model was to investigate whether there was any discernible difference in the performance of the structure is it was placed alone on a concave bend. In all other respects the model was identical to Model II. No measurable differences were observed.

4.7.4 Interpretation of the Model Results

Armour Sizing

The applicability of the relationship presented in the JMB Design Report has been verified for 40 and 60 cm size concrete cubes. It has been found that there is a satisfactory factor of safety built into this relationship (Figure 4.26). As explained above an adequate safely factor is required to allow for the scatter present in the stability performance of the armour blocks.

?? Mixtures of different sizes of blocks in the apron did not appear to behave measurably better than the single size alone. Use of single size blocks is therefore the preferred solution to facilitate construction and supervision.

The use of crushed rock as a grading "filler" (not a filter) for the launched armour layer did not provide any discernible advantage and there is a possibility that the larger spacing of the launched blocks could result in displacement at a lower velocity than would otherwise be the case.

This statement is again contradicting the statement " in page 4-28, 1st para last line

It is rather expected that there will be net movement of sand through the interstices of launched armour material if they are not properly graded

There was no indication of any net movement of sand through the interstices of the launched armour material. If material were being drawn out from the underlying bank by the action of pressure differentials then it would be expected that the resulting pattern of collapse of the slope would reflect the velocity variations down the length of the structure and between different configurations. The notably uniform form of the launched slope in all cases prior to failure, with and without fabric, therefore argues strongly against this possibility.

Armour Material

The majority of tests were carried out using simulated broken brick aggregate concrete cubes as the armour material. In one test rounded rock was substituted as the apron material and this performed equally well.

Velocity Distribution

With a simulated embayment upstream of the structure there was a strong concentration of flow around the upstream termination nose (see for instance the Sirajganj model in Figure 4.27). The highest velocity consistently occurred in an area starting from the transition from the conical termination to the straight section (Profile III in Figures 4.15, 4.16) and about 50 m downstream thereof. The highest velocity occurred well away from the revetment slope, and was concentrated in the region around and above the toe of the apron.

Although it is necessary to exercise caution when interpreting the observed velocities, some useful inferences may be drawn from a comparison of the maximum point velocities given in Table 4.8 and also for the ratios between maximum and mean velocities given in Table 4.9.

Table 4.8 Maximum Point Velocity with Embayment and without Embayment (Prototype values)

Profile	Q = 406 1/s (Model)		Q = 522 1/s (Model)	
	Without Embayment, Test 1B	With Embayment, Test 3B	Without Embayment, Test 1C	With Embayment, Test 2C
III	3.9 m/s	4.2 m/s	4.6 m/s	5.2 m/s
IV	3.8 m/s	4.2 m/s	4.5 m/s	5.2 m/s
V	4.1 m/s	3.9 m/s	4.6 m/s	4.9 m/s

Table 4.9 Maximum Near Structure Point Velocity in Brackets (V_{max}/V_{mean})

Test No	Discharge (l/s)	Maximum Near Bank Point Velocity m/s (Prototype)					
		Test Series III (normal placing)			Test Series VII (random placing)		
		Profile III	Profile IV	Profile V	Profile III	Profile IV	Profile V
B	406	4.2(1.7)	4.2(1.7)	3.9(1.6)	3.8(1.5)	3.7(1.5)	3.5(1.4)
B1	450	4.5(1.6)	4.5(1.6)	4.4(1.6)	3.6(1.3)	4.0(1.5)	3.9(1.4)
B2	480	4.9(1.7)	4.6(1.6)	4.7(1.6)	4.2(1.4)	4.1(1.4)	4.1(1.4)
C	522				4.5(1.4)	4.5(1.5)	4.4(1.4)

Note: Average Target Velocity: Test B, $\underline{V}$ = 2.50 m/s
Test B1, $\underline{V}$ = 2.75 m/s
Test B2, $\underline{V}$ = 2.93 m/s
Test C, V = 3.20 m/s

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For the purpose of selecting a velocity for sizing of the armour layer in any particular situation it is necessary to know the amplification factor to apply to the maximum near-bank velocity without the presence of the structure in order to compute the equivalent velocity after introduction of the structure (after bed adjustment has taken place). By comparing the maximum velocity in the vicinity of the revetment face in the model with that upstream of the structure it can be inferred that the amplification factor due to the concentration of streamlines around the protruding nose of the structure may be as high as 1.3.

Scour Depth

Table 4.10 shows the measured scour depth with and without an upstream embayment for two discharges, equivalent to target mean velocities of 2.5 and 3.2 m/s respectively. It can be seen that for the lower velocity there was some redistribution of depth between the two cases, greater scour occurring near the nose, but no overall change. For the higher velocity, however, there was consistent deepening along the length of the structure. The largest recorded water depth on a profile was 33.5 m at a distance of 10 m from the toe of the slope covered by the launched apron.

Table 4.10 Maximum Scour Depth (in terms of elevation)
(Water Level = + 29.5 m)

Profile	Q = 406 l/s (Model)		Q = 522 l/s (Model)	
	Without Embayment, Test 1B	With Embayment, Test 3B	Without Embayment, Test 1C	With Embayment, Test 2C
III	-7.5 m	-10.5 m	-6.6 m	-14.00 m
IV	-8.25 m	-8.75 m	-6.8 m	-13.10 m
V	-8.4 m	-4.80 m	-6.75 m	-13.00 m

Apron Geometry

The manner in which the armour material was distributed down the eroding slope appeared to be largely independent of the geometry of the apron. The more concentrated apron, attributed to the US Corps of Engineers, performed in very similar manner to that based on Indian practice.

The advantage of the wider apron is that it offers a better overall slope profile from the point of view of geotechnical stability. This has to be weighed against possibly lower construction costs associated with the more concentrated arrangement.

Apron Setting Depth

Although the model tests showed little difference between the manner in which the apron material launched for different setting depths, there are two strong arguments in favour of setting the apron at a lower rather than higher elevation. The first is that the eroded slope of 1V:2H is substantially steeper than the design slope of 1V:3.5H based on slip circle analysis. The higher the setting depth the greater the impact of this steeper lower slope on the overall bank stability. The second is that despite the favourable results of these tests, there remains an element of uncertainty as to the long-term stability of the launched apron layer in the absence of a filter layer to prevent migration of the underlying material.

Monitoring of actual structure performance will, however, provide more data giving better insight into prototype behaviour. It should be noted that at the present apron design setting depth the probability of exposure of the launched slope is not more than ten percent (i.e. the probability of exceedance of the equivalent water depth, measured below design water level, is less than ten percent).

This percentage is found as the probability of exceeding the design water depth of approximately 19.5 m below the 100 year flood level, or 16.5 m below dominant discharge level (see Figure 3.4). Figures 7.4 shows the probability of exceedance of maximum thalweg depth, based on surveyed cross sections. Typically this maximum depth will occur near one or other bank, and as this analysis concerns only the right bank, it is reasonable to reduce the probability by a factor of 0.5. The probability of water depth exceeding 16.5 m below dominant discharge can be seen to be approximately twenty percent, hence the probability of exposure of the launched slope may be taken as not more than the ten percent.

The importance of an effective filter layer on the upper part of the slope, where wave action and hydraulic gradients in the bank material are significant, is broadly accepted by designers of bank stabilisation works. It is also accepted that the only practicable way in which this filter can be provided is by means of a geotextile. It is probable that at depth the hydraulic gradients are lower, the bank material more permeable and wave action is negligible. The only forces tending to displace sand particles from below the armour layer are thus pressure differentials due to turbulent conditions at the face of the armour layer. The more evenly distributed the launched armour material the lower these differentials will be.

There is insufficient evidence at present to be certain about but the indications from the testing is that at levels below setting depth migration of sand in this way will at worst be a slow process, which can only take place when the launched material is exposed and the by passing currents severe.

It is also worth mentioning that the model tests have shown that only a portion of the apron will launch itself under normal conditions and there is consequently a large volume of sand that needs to be extracted/washed out through the apron material before the slope itself is endangered. This again indicates that the structure as designed has a substantial reserve of stability.

On the other hand there is good evidence that the presence of a textile layer under the apron results in a very much less uniform pattern of distribution of the launched material. This is in itself undesirable and it will also result in the exposure of relatively large areas of textile; it is known that such exposure leads to rapid deterioration due to abrasion and associated with flapping of the material. Once a hole develops failure of the slope will follow rapidly.

Until such time as contrary evidence may become available, the placing of geotextile under a falling apron should be limited to a sufficient width for the purpose of ensuring complete coverage under the fixed slope armour above the apron.

4.8 Alternative Forms for Training Works

As an alternative to conventional solid groynes, consideration has been given to the possibility of using permeable groynes in certain circumstances, particularly where the primary object is to reduce flow velocities in deeper water channels. An example might be the maintenance of a deep water navigation channel with an adjacent area of lower velocity currents for mooring. The design of the tests, which were carried out at an undistorted scale of 1:100, and the full results together with their interpretation will be found in Part 3 of the physical modelling section of the BRTS Report on Model Studies.

Three different arrangements were tested, all involving piles with a prototype diameter of 2.0 m. One consisted of a single row of piles set at 10 m centres, giving an effective porosity of 80 percent; a second with piles at 4.0 m centres giving a porosity of 50 percent; a third consisting of three staggered rows of piles set such that the distance between the centres of any two piles was 5.0 m, giving a porosity of only 20 percent.

The first of these structures had no measurable effect on the flow while the second reduced the velocity on average by about 10 percent and the third about 15 percent. Scour associated with these arrangements was closely related to their efficacy as velocity reducers. No detectable scour occurred with the first groyne whereas the second groyne produced a zone of about 20 m width with 8 m of scour and a maximum depth of scour of 10.4 m. Taking into consideration the starting water depth of 20 m, this would mean a total water depth in the prototype of over 30 m, which is of the same order as that associated with solid groynes. For a distance of about 180 m downstream of the structure there was fairly regular deposition of the order of 2 to 3 m. The third structure produced a similar pattern but with the general depth of scour being increased to around 12 m and the maximum depth increasing to 13.4 m; i.e. total water depth of 32m and 33.4 m. The associated deposition was between 2 and 4 m.

These structures were tested only as examples of possible solutions and have shown that in order to be effective in reducing the velocity on the lee side of the permeable groyne, the porosity should be low. This means an almost solid wall which again will result in even larger scour than found for the structure with 20% porosity.

There are therefore major structural problems associated with use of permeable groynes that will detract from their cost-effectiveness under the Brahmaputra conditions.

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4.9 Ferry Ghats

As an extension of the principles addressed in the previous section, consideration was given to whether the concept of embayments between groynes or hard-points could be developed as a means of providing a relatively stable length of sloping beach that would lend itself to the installation of simple but semi-permanent access arrangements for vehicle and railway ghats.

The principal requirements for a ferry ghat is that there should be a permanent deep water channel, preferably immediately adjacent to the ghat; it is also desirable that velocities in the vicinity of the berthing area should not be so high during periods of monsoon flow as to make mooring and manoeuvering difficult. Groynes in combination with hard-points are the conventional way of providing the deep water channel and the example of the natural bay that has formed between Ranigram Groyne and the Sirajganj revetment suggested a possible solution to the second requirement. The layout tested consisted of two 120 m long groynes separated by a distance of 540 m constructed at an undistorted scale of 1:100.

The full details of the model design, observations and interpretation are given in Part 5 of the physical modelling section of the BRTS Report on Model Studies. The arrangement produced a lagoon where the velocity was about 20 percent of that in the main stream, corresponding to about 0.5 m/s in the prototype, which is suitable for most river craft. The total area of this low velocity zone was about 400 m long and 100 m wide on average. The best deep water access was a channel running in to the lagoon from the downstream groyne.

The main interference to navigation took the form of a 300 m long and 60 m wide bar that developed; this ran from the upstream groyne in a gentle crescent shape.

More detailed work would be required if this arrangement were to be taken up but this preliminary model test serves to establish the principle and to show that the concept has validity.

4.10 General Conclusions and their Relation to the Master Plan

The three principal contributions of the physical modelling programme have been in relation to the prediction of scour depths, the determination of suitable design values for near-bank velocity and the investigation into the performance of revetment armouring and the falling apron. These are all of immediate importance to the design of the priority bank stabilisation works and also provide the essential basis for the design of longer term stabilisation and training works.

The extensive test programme has provided a large amount of information on scour depth prediction for a range of structural arrangements and the particular flow conditions encountered in the Brahmaputra. This has been checked with the limited data on actual scour that has developed in the vicinity of Ranigram and Kalitola Groynes and it has been found to be consistent. In the absence of a broader range of examples that can be observed in the field, physical modelling offers the only means of carrying out such an investigation. The conclusions in terms of design scour depths for various types and layouts of structures are presented in paragraph 4.5.5 and Appendix B.

The second major contribution has been the quantification of velocity distribution around the nose of groynes and the upstream terminations of hard-points. This has been carried out in conjunction with 2-D modelling studies and the results have been verified by comparison with field measurements taken by the BRTS river survey teams.

Thirdly but not least the testing carried out at the (scale of 1:50 has provided unique insight into the performance of the revetment and the all important falling apron. Once again, no other approach but prototype observation can provide this type of information. The series of tests has raised the level of confidence in this form of protection when utilised in the severe conditions found along the banks of the Brahmaputra and provided a rational basis for apron sizing. Of equal importance has been the direct verification of the relationship proposed by the JMB study for the sizing of armour material.

5. RIVER HYDRAULICS AND CHANNEL CHARACTERISTICS

5.1 Objective and Scope of the River Survey Programme

The objective of the river survey programme was to collect input data for the mathematical and physical modelling, geomorphological studies and the design of river engineering works.

Activities included an extensive collection of bathymetry data at several sites along the Brahmaputra River and also current velocity measurements and sediment sampling at two sites selected for investigation as part of the 2-D hydrodynamic modelling programme.

The overall structure of the programme is illustrated in Figure 5.1 an the locations at which surveys were carried out are shown in Figure 5.2.

The scope of the data collection undertaken and a summary of the findings is presented in the Appendix D. The figures in Table 5.1 gives some indication of the scale of the task.

Table 5.1 Summary of Data Collection During River Surveys

Item	Quantity	Unit
Level Traverses For Bench Marks	190	km
Level Traverses For Topographic Surveys	80	km
Plane Table Surveys	330	ha
Bankline Surveys	15	km
Bankline Movement Surveys	25	km
Bathymetry Traverses (Test Area 1)	270	km
Bathymetry Traverses (Test Area 2)	150	km
Bathymetry Traverses (Priority Sites)	165	km
Velocity Readings	231	no
Float Tracking Traverses	40	km
Suspended Sediment Readings	378	no
Dune Tracking Bed Samples	120	km
Bed Samples	102	no

A full description of the work carried out under the river survey programme is given in Annex 1 (Parts 1 to 4) to the First Interim Report and Annex 1 to the Second Interim Report.

5.2 Analysis of Velocity Data from Test Area 1

Two sections of the Brahmaputra River were selected as providing in combination suitably representative conditions for calibrating and verifying the 2-D modelling system. These are referred to as "Test Area 1" and "Test Area 2". The former covers the full width of the river over a 12 km long reach south of Sirajganj and contains both major confluence and bifurcation features. The second covers the western anabranch in the vicinity of aipur and was chosen for its very pronounced bend scour associated with strong helical flow currents which were not so evident in Test Area 1.

The surveys were carried out in June, July and November 1990 in Test Area 1 and January and August 1991 in Test Area 2. The data collected thus covers the five distinct stages in two monsoon season nd the intervening low flow period. The Test Area 1 data was used to compute river discharges across ten sections spaced at approximately 1 km intervals.

Since the data used for the discharge computation in Test Area 1 were all collected during the space of a week, during which water levels rose by only 30 cm - signifying an increase in discharge barely within measurable accuracy with the methods and equipment conventionally employed - these measurements could be treated as having yielded ten samples of discharge measurement from a consistent and reproducible process.

A statistical analysis of the samples showed that all measurements fell within the 95% confidence limits of a mean discharge, the standard error of which was about two percent (2%) of the mean discharge. The discharge in the Brahmaputra, for the week during which the river survey was carried out, was thus could be estimated to be 42,000 m^3/sec with 95% confidence limits.

The BRTS survey was the first ever monsoon season river survey of the Brahmaputra-Jamuna for the collection of contemporaneous sets of bathymetric, velocity and sediment data. The experience gained from the survey covered many facets, including the limitations of the various types of equipment used. Perhaps the most useful outcome of the survey was that the estimate of discharge in the Brahmaputra would need to be based on at least four consecutive measurements if it is to be determined within a standard error of less than 2,000 m^3/s, using equipment equivalent to, or better than, that employed in the BRTS survey.

The discharge measurements obtained were the outcome of bathymetric and velocity measurements carried out between the 12th and 19th July 1990. To approach contemporaneity, velocity measurements in a cross section were taken within 24 hours of the bathymetric survey of that river section.

The purpose of the river bathymetry and flow velocity measurements was primarily to obtain calibration data for 2-D mathematical model. It was appreciated, however, at the time of design of the monsoon season river survey campaign in Test Area 1, that the most comprehensive internal check on the accuracy of measurements (bathymetry and point velocities) would be a comparison of the discharges which could be computed on the basis of such measurements.

This was perceived to be particularly appropriate as measurements were to be made across ten river cross sections thereby permitting the comparison of discharges computed using ten independent data sets. Thus, even if it was of secondary importance to the requirements of the 2-D model itself, the computation of river discharges was clearly identified as the most comprehensive check on the river survey. Consequently, the data processing procedure (see Figure 5.3) was designed with this requirement in view.

Bathymetry

The accuracy of measurement of the area of flow is dependent on the accuracy of measurement of (a) the depths and (b) the distance to each depth measurement from a known point on the section. Both depth and distance should ideally be measured as continuous (as opposed to discrete) values so as to minimise interpolation between points of measurement. In the BRTS measurement, depths were recorded continuously across a section by an echo-sounder but distances were fixed on the echogram at 1-minute intervals during the traversing of the cross section; this resulted in a "fix" at intervals of about 100 meters. Position fixing was by Decca navigation system which provides good reception conditions for position-fixing in Test Area 1.

Velocity

The measurement of velocity was carried out using a Braystoke directional current meter carried on a Valeport winch. The locations at which velocity measurements were to be taken were decided by studying the echogram for each cross section and their Decca co-ordinates then obtained from the format on which the bathymetry of the section was recorded. Current meter readings were taken generally at 0.2 and 0.8 times the total depth.

Water Levels

The reduction of the bathymetric levels to the BWDB datum, the datum adopted by BWDB for all its work, required the recording of water levels during the period of the river survey related to the PWD datum.

The water level gauge maintained by BWDB at Sirajganj is an important station in its network and was thus adopted as the upstream water level gauge for Test Area 1 ; a site on the right bank located immediately to the south of Test Area 1 was selected to provide water level data at a point far enough from Sirajganj to register an adequate drop in water level between the two stations whilst also just spanning the area of survey to enable satisfactory interpolation of water levels within Test Area 1. The site selected was at Shahpur/Deluachar, some 17 km south of Sirajganj.

Discharge

Discharge was computed from area of flow (from water level and bed topography) times depth averaged velocity for each "flow panel" within the cross-section, as shown in Figure 5.3.

The maximum depth sounded in Test Area 1 was 20 m and the maximum velocity was 2.34 m/s. As noted above, the discharge was estimated at 42,000 m^3/s.

Sources of Error.

Discharge in a river is calculated as the product of velocity and area, measurement of each of which has its own inherent errors. velocity measurement at a point by current meter is influenced by instrumentation error, and the average velocity in a given vertical depends on the relationship used in calculating it. Similarly, area measurements are dependent upon errors inherent in sounding and position fixing.

Established procedures, however, assure a generally accepted degree of reliability of streamflow data in many water courses. Nevertheless, in a river such as the Brahmaputra, even established procedures need to be reviewed due to two major consequences arising from differences in the morphological characteristics of the Brahmaputra from rivers in which such instruments and methods are generally used. The first is the difficulty of position-fixing of the point of measurement in a large braided river where there is no inter-visibility between the two banks. The second is the effect of rapid changes in river bed topography and the speed with which the measurements need to be taken to ensure contemporaneity of the data sets of bathymetry and velocity at any cross section. The possible sources of error in the use of such procedures for discharge measurements of the Brahmaputra are set out in a 'tree diagram' in Figure 5.4.

The main source of error in the BRTS measurement is thus that arising from the velocity measurements not being adequately representative of the flows across the section. Careful selection of verticals, the number thereof, and the number of cross sections measured have, however, helped to keep reduce the magnitude of this error within the limits described above.

5.3 Temporal and Spatial Velocity Variations

In any river there is a variation in velocity vertically due to boundary shear effects, horizontally in a lateral direction due to helical flow resulting from centripetal forces, and longitudinally due to changes in channel geometry. In a reasonably stable meandering river the velocity at any point in the river will, for a certain discharge, tend to be relatively consistent in magnitude and direction over time (setting aside the hysterisis effect common to sand bed rivers). In a braided river such as the Brahmaputra the velocity is far more variable. Not only is the thalweg constantly shifting in planform in response to varying flow conditions but also the migration of the macro sandbars is superimposing additional variable constraints on the direction of flow and the cross-sectional area of the thalweg. Furthermore, the confluence of flow streams downstream of a char can result in zones of unusually high velocity and turbulent conditions. The combination of these effects produces a much larger spatial and temporal scatter of velocity than would be expected in a meandering river. Moreover, it is not possible to predict by any deterministic means what the velocity will be at a particular point in the river for any given discharge.

Since the sizing of slope protection constituents is very sensitive to the velocity selected for design purposes (size being a function of the square of velocity), it is desirable to establish as accurately as possible the relationship between velocity and non-exceedance probability for any point where structural works may be implemented. Since there is insufficient measured velocity data available to carry out a statistical analysis that could be considered representative of the river as whole, three different approaches were used in order to arrive at a distribution function for the mean sectional velocity. This was then combined with a function expressing the variation across a section to arrive at a combined probability relationship that could be used to select a suitable near bank velocity for the design of Phase 1 bank stabilisation works. The full procedure is described in detail in Appendix C.

Analysis of the velocity measurements carried out in Test Area 1 during the BRTS river survey programme provides an additional check on this relationship.

As discussed at the end of section 3.5.2, significant water surface elevation differences can occur across the width of the river.

When?

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5.4 Seasonal Channel Geometry Changes

The bathymetry has been surveyed three times as a part of the Test Area 1 survey.

June 18th to 26th, 1990
July 9th to 18th, 1990
November 13th to 26th, 1990

Full details of the cross section characteristics are given in Annex 1 to this report.

Comparison of the findings in Test Area 1 with other investigations reveals some discrepancies.

The central part of Test Area 1 is a so-called nodal point, where the main anabranches meet. According to Coleman (1969) such a section should have larger mean depth than other sections of the river. However, the surveyed sections around the crossing are not in general deeper than the remaining sections.

Coleman also reports a persistent pattern in the cyclic development of the cross-section.

"As discharge increases, two channels, generally located adjacent to the river banks, tend to scour, while the central part of the channel is the site to deposition. As flood waters subside, one channel tends to fill rapidly, forcing water down the other, thus maintaining a single deep channel. After the flood has passed, little change takes place except for some filling and reduction in cross-sectional area of the main channel.

This pattern cannot be verified with the Test Area 1 data:

- both erosion and deposition take place in the central part of the cross-section
- there is no general increases of the main channels adjacent to the banks from June to July.
- from July to November there is no significant decrease of the minor channels.

RPT/NEDECO/BCL (1989) reports that the scour depth (bend and confluence) does not vary systematically with stage, (the inference being that scour statistics can be derived from dry season data. The Test Area 1 data shows that some scour holes increased in depth between the flood season and the dry season whereas others decreased. This does not conflict with the assumption that dry season data can be used to derive scour statistics but does demonstrate that the location of maximum scour is a dynamic function.

5.5 Sediment Transport Characteristics

5.5.1 Sediment Size

The Brahmaputra's catchment supplies vast quantities of sediment from the actively uplifting fold mountains in the Himalayas, the erosive foothills of the Himalayan Foredeep and the great alluvial deposits stored in the Assam Valley. Consequently, the Brahmaputra in

Bangladesh carries a heavy sediment load, estimated to be over 500 million tonnes annually. Most of this is in the silt size class but around 15 to 25 percent is sand. Owing to the geology of the catchment, the clay fraction is very small.

The sand size sediment is relatively uniformly graded as illustrated by a typical particle size distribution shown in Figure 5.5. In all 62 samples were taken from both the river bed and from sandbars by BRTS staff and tested by the RRI at their Faridpur laboratory. Typical grading curves for material sampled from the floodplain and higher levels chars (islands) are also shown for comparison (Figure 5.6). There is reason to believe that the bed material does become slightly finer in a downstream direction although the limited results currently available do not provide conclusive evidence of this. For the purposes of this study the range of d_{50} values has been taken as lying between 0.21 mm at Chilmari to 0.16 mm at the Ganges confluence.

5.5.2 Sediment Transport

The silt fraction, often referred to as wash load, is carried in suspension by the river and it is believed that most passes through the study reach, with only a small proportion becoming deposited in the lower velocity zones on char tops and floodplain during the limited period when the river level is higher than bank level. The silt load is not therefore considered to be important in terms of channel morphology, although probably significant in relation to char morphology, particularly during the final stages of char top building; a conclusion that is consistent with experience in other countries.

The sand fraction is transported through a combination of true bed load, which probably represents about 10 percent of the total sand fraction movement, and a complex pattern of partial suspension that occurs mainly in the zone close to the bed but is seen at higher flows in the form of dramatic "boils" in which the heavier fractions are carried temporarily to the upper levels. This second process is closely linked to the movement of both the smaller dunes and the massive sandbars that are a dominant feature of the braid pattern. Results of dune tracking undertaken in June 1990 indicate that the former were about 3 m high and perhaps 200 m long and their migration rate was found to be of the order of 35 m/hour. The larger sandbars vary in size but most lie in the range of from 3 to 6 km in length and 1 to 2 km in width; they are typically drowned at flows of between 30,000 and 40,000 m^3/s and are capable of travelling downstream during the monsoon season at around 30 m/day, although when associated with an actively eroding bend they may scarcely move for several seasons. There is evidence that these massive sandbars are the primary manifestation of the sand fraction transport.

5.5.3 Sediment Transport Relationships

Data on coarse (i.e. suspended bed load material and excluding wash load) suspended sediment transport in the Brahmaputra River at Bahadurabad are available for the period 1968-70 and 1982-88. The observed suspended sediment transport plotted against discharge is depicted in Figure 5.5 together with a regression line reading.

$$Q_{susp} = 9.1\ 10^{-6} \times Q^{1.26}\ (t/s)$$

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A closer analysis reveals that the data from the period 1968-70 suggest a significant (about 3 times) higher sediment transport than suggested by the 1982-88 data. The data are depicted in Figure 5.6. The regression lines are

1968-70:

$$Q_{susp} = 4.97\ 10^{-6}\ Q^{1.38}\ (t/s)$$

1982-88:

$$Q_{susp} = 15.2 \times 10^{-6}\ Q^{1.184}\ (t/s)$$

It is most likely that the apparent increase in sediment transport is caused by either a change in measuring procedure or a change in data processing. An increase of sediment transport with a factor 3 normally would imply an increase of velocity of 30% to 40%. This could only be achieved if very significant morphological changes had taken place, and there is no indication of such morphological changes.

The following is noted:

(a) The high flow (July) data from Test Area 1 suggest a slightly higher transport rate than the 1968-70, although the November data agrees better with the 1982-88 data. It should be mentioned that the Test Area 1 data only represents two sets of observations of the total sediment transport, which is subjected to natural fluctuation due to migrating bars etc.

(b) Most sediment transport models will predict transport rates which agree better with the 1968-70 data than with the 1982-88 data, see Figure 5.7.

The 1968-70 data probably therefore provide a better description of the conditions in the Brahmaputra River, particularly during the monsoon season when the large majority of sediment transport takes place. This is consistent with the relations applied in the JMB feasibility study.

Fall velocity estimated from ASCE Sedimentation Engineering (1975) and a water temperature equal to 20°C. This suggests a fall velocity equal to 2.0 cm/s.

A number of sediment transport formulae have been applied to attempt to describe the sediment transport in the Brahmaputra.

Based on dune trackings, it is apparent that dunes in the Brahmaputra migrate much faster than would be expected from any known bed-load formulae. For this reason, it is necessary to estimate how large a proportion of the suspended sediment transport is deposited in the lee of the sand dunes and activity contributes to their movement. It is concluded that a substantial proportion (more than 50%) contributes to the dunes movement.

Two models, the Engelund-Fredsoe and the van Rijn models, have been used to calculate the sediment transport in the Brahmaputra. The advantage of these two models is that they are able to distinguish between bed load and suspended load, which is essential for mathematical

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modelling of bend scour. The two models predict slightly larger transport rates than the Engelund-Hansen model, which is a total load model.

However, the Engelund-Fredsoe model and the van Rijn models correspond very well with the measured field data.

5.6 Conclusions

Velocity measurements have been taken under the BRTS river survey programme and used to provide data for the calibration and verification of the 2-D modelling system. In conjunction with the measured bathymetry, these results have been used in the calculation of discharge. Velocity measurement is inherently error prove and if reasonable accuracy is to be achieved then at least four cross sections should be analysed.

Velocity distribution plays a major part in the design of river training structures. In a river such as the Brahmaputra it is highly variable and cannot be predicted by deterministic means. Insufficient measured velocity data is available to be considered representative of the river as a whole, and a probabilistic approach has been adopted to determine the near bank velocity used in design.

The silt fraction, or wash load, is carried mainly in suspension and is not considered important in terms of channel morphology. The sand fraction is transported by a combination of true bed load, dune movement and the movement of massive sandbars; evidence suggests that the latter is the primary means of sand fraction transport.

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6. RESULTS OF THE MORPHOLOGICAL STUDIES

6.1 Objectives and Scope

The morphological studies cover a wide spectrum of activities aimed at developing an insight into the character of the river and establishing the links between channel form and the principle processes involved. The end objectives being, firstly, the development of guidelines for the prediction of behaviour patterns such as the evolution of aggressive bends and the medium and longer term movement of the bankline, secondly, the derivation of values for the key parameters required for the design of bank stabilisation and other river training works.

The studies have been planned and executed as one element of a fully integrated study programme which has also made extensive use of physical and mathematical modelling components. The interdependence of these several planning tools will be apparent from the description of activities given in this chapter.

6.2 Data Sources and Quality

6.2.1 Data Sources for Morphological Studies

The sources of data available for the BRTS analysis of river planform characteristics are listed in Annex 2. The main categories are:

(a) Survey of Bangladesh 1:50,000 scale topographic maps published in 1951-57, 1967-69 and 1978-79. These are considered to be the most reliable record of bankline for the period prior to the commencement of satellite image coverage and are in ready to use form.

(b) Rectified photographic prints at 1:50,000 scale obtained from the November 1989 SPOT satellite imagery.

(c) Partial coverage of the area by photographic prints of 1:50,000 and 1:20,000 scale aerial photography flown in December 1989 by Finnmap. This provides a potentially valuable complement to the SPOT imagery for interpretation of features which are not well defined on the former. Its use is limited by the security restrictions imposed by the Ministry of Defence and coverage for certain key areas, such as Sirajganj, is not available.

(d) Good quality copies obtained from the India Office Map Room, London, of the Rennell map of 1765, the Wilcox map of 1830 and the Survey of India map issued in 1914, the latter two at a scale of 1 inch to 4 miles. The 1914 map is a detailed topographic map showing a large number of villages and features in a similar style to modern maps and it may be assumed that its reliability with regard to bankline planform and major char outline is of a high order. The Wilcox map covers only the river and a narrow strip on either side; it is drawn to a high standard with good detail and referenced to longitude and latitude. Cross-reference to villages on modern maps is satisfactory and there is no reason to presume that the map is not reliable with regard to planform. The Rennell map is also well drawn and detailed; it is not referenced to longitude and latitude but after adjustment to a common scale it is possible to relate it satisfactorily through

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village and town locations and thereby establish a common reference with the other maps. The outline of the river fits well with major bend scars and other features identified on the 1:50,000 SPOT imagery and it is reasonable therefore to accept it as a reliable record of the location of the main banklines at that time.

(e) River cross-sections surveyed by the BWDB Morphology Division between 1964 and 1989 with a break from 1970 to 1976. These were mainly received in the form of rather indifferent quality ammonia prints and were digitised in the BRTS office.

(f) Landsat MSS imagery for February 1973, January 1976, February 1978, February 1980, February 1984 and February 1987, and TM imagery for January 1990 and March 1992 which has been rectified, enhanced and processed by ISPAN under the FAP-19 project. This is high quality data on which much of the quantitative analysis has been based.

(g) Photographic prints at approximately 1:250,000 scale of unrectified Landsat imagery for February 1973, January 1976, January 1977, February 1978, February 1980, December 1981, March 1984, March 1985, March 1986, February 1987 and February 1988.

(h) SPOT imagery at 1:50,000 scale for November 1989, March 1990 and November 1990 which has been registered to the same grid and sheet specification as used by the survey of Bangladesh. Although this has a higher resolution than the landsat imagery, the limited time period coverage means that it is of less value for morphological studies.

6.2.2 Data Quality Verification

The rectified satellite imagery provides the most reliable database but has been available only since 1973, with data obtained at the times noted above. ISPAN estimate that after rectification ultimate accuracy (horizontal) based on pixel size is 80m for landset MSS, and actual accuracy is 2 pixels. The pixel size for landset TM is 30m, but in the imagery used this has been resampled at 80 m for consistency with the earlier MSS imagery. Some practical difficulties have been experienced over the definition of banklines but these relate more to the interpretation of what constitutes the bankline of a complex braided river than any limitations inherent in the imagery. This level of accuracy has not been a significant constraint on the outcome of the studies.

Some doubts have been raised about the reliability of the Wilcox and Rennell maps, particularly the referencing to latitude and longitude. In the case of the Wilcox map the latitude and longitude shown on the map were ignored and the registration was based on identifiable persistent physical features. Although some uncertainty must remain, it is considered that the map provides a reliable picture of overall planform and, with due reservation, can be used for qualitative assessment of the history of the river following the major avulsion that took place at the end of the 18th century.

It had been anticipated that the river cross-sections, that have been regularly surveyed by the BWDB since 1964, would provide an excellent record of the river from which a large amount of information on the channel form characteristics could be derived, including an indication of any trend over time. Accordingly all available sections, which were plotted to a variety of

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scales, were carefully digitised and the very large amount of data entered into a database for further processing. After plotting out to a common scale the data verification included comparison of sections between sequential years and cross-checking where possible against satellite imagery for consistency of planform. Concurrently with this exercise, field surveys were being carried out to determine the current position of the reference monuments, many of which were several kilometres away from their positions as recorded on the location record map. The reason for this is that when monuments are eroded by the river, they are re-established at a safe distance from the river and the datum levels are transferred but the new position is not routinely determined by survey. Thus while it is possible to fix the current positions it is not possible to determine the exact location of intermediate positions.

The comparison of sequential plots indicated that substantial datum shifts, both horizontally and vertically, have taken place but it was not possible to establish any systematic pattern that could be adapted in order to make corrections. Interpretation of the cross-sectional data must therefore be made with the full knowledge of these inherent errors, which are at their most significant in the horizontal plane (see Annex 2 for further details)

6.3 Brief History of the River

The earliest reliable information on the planform of the river is probably the map published by Rennell in 1765. This shows a braided river hugging the Shillong hills and then taking a south-easterly alignment to follow the eastern edge of the Madhupur Forest tract, which is a slightly elevated area of more resistant Pliocene deposits. Somewhere in the vicinity of Mymensingh the river changed to a much more meandering pattern of low sinuosity (Figure 6.1).

The geomorphological map published by Coleman (1969) shows that the Brahmaputra had previously followed courses further to the north-east and had trended south-west to its 1765 course. Given this trend and the constraint to further movement provided by its contact with the harder Madhupur material, the river was clearly poised for a major channel shift, or avulsion. The two most favoured direct causes of the avulsion are tectonic movement and liquefaction following a major seismic event; it seems likely that the former is responsible for the underlying trend while a liquefaction flow that partially blocked the old channel could feasibly have been the trigger that initiated the avulsion. The details of the process, and the precise timescale, may never be fully known and are probably of only secondary importance with respect to prediction of future trends. Unless the tectonic pattern changes substantially it would seem unlikely that the river would be inclined to return to its old, and considerably longer, course in the foreseeable future. A slow continuation of the westward migration, until the river encounters the harder material of the Barind tract forming the western edge of its valley, appears to be a more probable scenario.

Other less reliable maps from the period show the main Teesta river taking a more westerly alignment than today but a left bank distributary channel marks the course of the present river. Suggestions that the addition of the Teesta flow to that of the Brahmaputra was a cause of the avulsion cannot be substantiated. Also shown on these maps are north-south aligned distributaries leaving the right bank of the Brahmaputra in the vicinity of the present day Bahadurabad; these would have provided pilot channels for the new Brahmaputra course and there seems little need to look further for an explanation of the avulsion process.

The map published by Wilcox in 1830, which was drawn up from a systematic triangulated land survey, is considered to provide a reliable picture of the planform of the river a few years after it had effectively completed its avulsion. Bend scars that closely match the form of his mapped bankline can be easily discerned on the SPOT imagery and there is no reason to doubt the veracity of the map in this respect.

The most distinctive feature of the Wilcox map is the fact that the river south of the avulsion point has an almost pure meandering single thread planform, whereas further north the braid pattern remains strongly pronounced. The underlying straight alignment, corresponding not only to the shortest course but also perhaps linked to the pilot channels mentioned earlier, is also noteworthy in terms of the river's macro-planform evolution.

By 1914 the major meander loops had broken down through a process of multiple chute channel development to create a largely braided appearance, but with a distinctive long straight reach north of Sirajganj. Because of the fundamental changes taking place it is hard to compare these two planforms in terms of migration or bankline movement but it would appear that some creep westwards had already been initiated.

More modern maps based on aerial photography are available for the period 1951-57 and by this time the river had taken on a planform that is generally recognisable today, with island clusters beginning to develop in approximately the same locations as the current vegetated islands (chars).

The present day planform would appear to be approaching the fully developed pattern with major islands and their satellites as illustrated schematically in Figure 6.2. This may be compared with the actual island pattern shown in Figure 6.3. If this model is indeed applicable, one may expect that the evolution over a timescale of the order of 30 years will consist of further widening of the braid belt as secondary islands grow, superimposed on a general tendency for the river to migrate slowly westwards. The highly braided reach of the river located in India a short distance upstream of the border is a model of how such multiple island building can develop; perhaps more likely however is a migration based on the assimilation of chars into the floodplain on one bank of the river. An example of the latter was the attachment of a large island group, located to the south of Sirajganj, to the left bank between 1950 and 1960. A similar process involving the two large islands opposite and to the north of Sirajganj may be in progress today.

The evolution of the river in its new course is consistent with the theory proposed by Bettes and White (1969) which, much simplified, is based on the thesis that a river has a preferred slope which it achieves by meandering if the valley slope is sufficiently flat, but if the valley slope is steeper it can only achieve stability by splitting into one or more anabranches and taking up a braided pattern. The fact that the river passed through the meandering stage and entered the braiding mode, where it has remained since, is a strong indication, though not conclusive, that it is now in its preferred state.

The old bend scars on the left bank flood plain corresponding to the planform shown on the maps of the early 1900's, suggest that the pattern of large bends has been a persistent feature of the lower section of the river. It is possible that the backwater effect of the Ganges, which results in some reduction in average water surface gradient, may have some influence on the planform in this reach.

This apparent tendency for the river south of Sirajganj to display a more meandering form is supported by the analysis of braiding indices for the period 1973 to 1990, but there is no evidence that it is becoming more pronounced. On the contrary, the satellite interpretation completed by ISPAN under the FAP-19 programme shows the steady growth of persistent major midstream islands in this reach, linked to very high sustained bank erosion rates, which points strongly to a long-term increase in braiding. The relatively higher widening tendency of this stretch of river since 1973 can be seen on Plate 16.

6.4 Dominant Discharge

6.4.1 Determination of the Dominant Discharge

The dominant discharge principle first proposed by Wolman and Miller (1960) is based on the concept that in rivers which experience a highly variable range of flows, the long-term average dimensions and geometry of the channel are determined by the flow which performs the most work, where work is defined as sediment transport. Although Wolman and Miller's concept has frequently been questioned on theoretical grounds, few people question its usefulness and validity as an analytical device in the geomorphological assessment of rivers.

In applying the principle to the Brahmaputra River, daily discharge data for the period 1956/57 to 1988/89 for Bahadurabad gauging station were used to construct a flow frequency distribution relationship. Sediment transport measurements taken between 1968-1970 for sand load and 1982-1988 for both sand and total suspended load were used to construct sediment rating curves, giving the following relationships:

1) Total sediment transport (1982-88 data)

$$Q_{st} = 0.91\ Q^{1.38}\ \text{t/day}$$

2) Suspended sand transport (1982-88 data)

$$Q_{ss} = 0.93\ Q^{1.25}\ \text{t/day}$$

3) Suspended sand transport (1968-70 data)

$$S_j = 4.1 \times 10^{-6}\ Q^{1.38}\ \text{m}^3\text{/s}$$

The last equation was tested specifically to address the fact that this is the curve favoured in the JMBA study and the one preferentially used in the present BRTS study for morphological modelling, even though it is based on a relatively short period of record, on the grounds that the data quality is considered to be better.

The flow frequency curve was divided into discharge classes with increments of 5,000 m^3/s and the frequency of each class was multiplied by the appropriate sediment transport rate, to produce a total sediment load transported by that discharge class during the period of record (Figure 6.4). This was repeated for the different sediment relationships.

Examination of the total sediment transport distributions in Figure 6.4 shows that the choice of sediment rating has little impact. The dominant discharge is defined by the analysis to be

38,000 m^3/s. The much smaller, secondary peak is associated with a discharge of 7,500 m^3/s, which corresponds to base flow for the river.

This figure also agrees with the dominant discharge quoted in the Study Report by the China-Bangladesh Joint Expert Team (CBJET), (1991) of 37,500 m^3/s , which was derived in a similar fashion. Examination of the flow duration curve for Bahadurabad (Figure 6.5) indicates that the dominant discharge is equalled or exceeded 18 percent of the time and that it is always exceeded during the high flow season. These findings are consistent with results for other large rivers.

6.4.2 Cumulative Sediment Discharge

The cumulative sediment transport curves for the three different sediment rating curves were derived by progressively accumulating the sediment loads for each discharge class. The resulting curves are very similar (see Annex 2 for details); all show the distinctive "S" shape with a very steep, almost linear increase in cumulative sediment load for discharges between 32,500 and 50,000 m^3/s. Thus flows in a range of only 17,500 m^3/s, are responsible for transporting about forty percent of all the sediment moved by the Brahmaputra River and the cumulative contribution of all floods greater than 50,000 m^3/s is less than about 20 percent of the total load, while that of flows greater than 70,000 m^3/s is less than 2 percent. The return period for 70,000 m^3/s is only about 3 years.

These results highlight the importance of flows around the dominant discharge in forming the channel, and identify that high, but in-bank, flows between 25,000 and 50,000 m^3/s have a disproportionate impact on channel form since they transport about half of the total load.

6.5 Char and Bar Inundation

6.5.1 Braid Bar Inundation

There is ample evidence from rivers with a wide variety of bed material that, for rivers in the meandering class, dominant flow is closely related to bankfull flow in terms of discharge magnitude (Richards, 1982) and (Knighton, 1984). In multi-threaded and braided rivers this is not thought to the case (Lee and Davies, 1986; Biedenharn et al, 1987) and in fact dominant discharge is expected to be less than bankfull discharge in braided rivers.

Perhaps the most prominent features of a braided river are the major sand bars, exposed during periods of low water level, which are responsible for the river's characteristic multi-channel cross-section, very high width/depth ratio, braided planform and shifting nature.

A visual examination was made of the water surface elevation corresponding to dominant discharge in relation to char top elevations at all river cross-sections surveyed in 1988/89. Water surface elevations were taken from a preliminary run of the BRTS version of the Mike 11 1-D General Model, for a flow of 38,000 m^3/s. The results are shown in Figure 6.6 (see Annex 2 for more details)

Two distinct char top levels may be discerned along much of the course of the river. A typical cross-section. (Figure 6.6) shows a higher char at close to bankfull elevation and lower chars at a little less than dominant flow level. Practically in all cases, the dominant flow inundates

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the lower chars but does not overtop the upper chars. On this basis, dominant flow may be seen to correspond to char topping discharge of the lower chars. For the upper chars to be inundated requires "bankfull" flow of perhaps 60,000 m^3/s, compared to the JMB study estimate of 45,000 m^3/s.

In this respect, the lower chars are "adjusted" to the present dominant flow and are contemporary morphological features. The upper chars, or islands, divide the flow even at dominant discharge and their tops are inactive except during high magnitude, out-of-bank flow events. The upper char, or island, surfaces are in this sense much more similar to fragments of contemporary flood plain within the braid belt then they are comparable to the contemporary chars.

Comparing the cross-sections with the March 1989 SPOT imagery showed a very good correlation between the upper chars and the mature vegetated areas, providing confidence in the general reliability of the cross-sectional data.

Closer examination of the lower char elevations reveals a stepped profile with longer, steeper reaches where the elevation difference between the chars and islands is a metre or more and chars are well inundated (Figure 6.7), and shorter, flatter reaches where the difference is much less and the chars are barely inundated at dominant flow. Preliminary investigation of the locations of the steps may be related to planform width and stability variations. At this stage it should be noted that "island reaches" show a clearer separation of upper and lower char surfaces than the intervening cross-over reaches.

As a further test of these relationships between discharge and sandbar and char level, the results of the 1-D hydrodynamic 25 year run were analysed to determine the frequency at which different water levels were exceeded and these were then compared with the sandbar and char levels shown in Table 6.1. It was found that the water level having 100 percent probability of exceedance for a notional duration (i.e the elevation of land that will be inundated for at least one day every year) corresponded extremely closely with char (island) top level (Figures 6.8 and 6.9). By back analysis it can therefore be inferred that the water level corresponding to dominant discharge has close to 100 percent exceedance probability.

6.6 Bank and Char Sediment Exchange

Conceptually, the sediment that is produced through the process of bank erosion may be treated by the river in one of three ways:

(a) transported out to the Bay of Bengal;

(b) carried some distance before being deposited in a mobile bed form;

(c) transported some distance before being deposited as long-term storage in an upper level meta-stable char.

If the river is in dynamic equilibrium as indicated by the specific gauge analysis and the interpretation of the long section data, then it would be expected that the bed and suspended sand load entering at the head of the river reach should be equal to that leaving at the bottom of the reach, when measured over a reasonable period of time. Although this need not be true

Table 6.1 **Relationship between Bank, Bartop and Chartop Elevations**

Cross-Section Number	Reference Chainage (km)	Water Surface Elevation	Mean Bank Elevation	Bartop (Active Char) Elevation	Chartop (Inactive) Elevation
J-17	25.00	23.7	24.6	22.0	23.5
J-16	31.35	22.6	24.0	21.0	23.4
J-15	44.25	21.2	23.0	20.8	22.5
J-15	55.65	20.5	22.0	19.0	21.5
J-14-1	63.00	19.9	21.4	18.2	21.0
J-14	71.00	19.1	20.6	18.3	20.0
J-13-1	81.70	18.6	19.8	18.0	19.5
J-13	84.70	18.4	19.8	17.5	19.0
J-12-1	93.80	17.7	19.1	17.6	18.8
JN-2	95.50	17.6	18.9	17.3	18.8
J-12	100.50	17.3	18.6	17.0	18.1
J-11-1	108.90	16.8	18.0	16.8	17.5
J-11	117.75	16.0	17.2	14.8	16.8
J-10-1	126.50	15.5	16.6	14.6	15.8
J-10	134.30	14.9	15.7	14.6	15.5
J-9-1	139.00	14.5	15.2	14.4	14.9
J-9	142.45	13.9	14.9	13.8	14.8
J-8-1	145.40	13.6	14.6	13.5	14.3
J-8	149.50	13.3	14.2	13.8	14.0
J-7	162.35	12.4	13.4	13.0	13.0
J-6-1	170.75	11.8	13.0	None	12.5
J-6	177.70	11.0	12.5	11.3	12.0
J-5-1	180.60	10.9	12.2	11.0	11.0
J-5	188.20	10.3	12.0	9.8	11.7
J-4-1	195.75	9.7	11.4	9.1	10.7
J-4	201.30	9.3	11.0	9.0	10.2
J-3-1	205.15	9.2	10.6	8.6	10.55
J-3	213.20	9.1	10.1	8.5	8.5
J-2-1	220.00	8.9	9.9	8.0	8.0
J-1-1	229.40	8.5	8.8	5.0	None

Note: All Elevations are in metres above PWD datum.

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for the silt load which has little impact on the river morphology and whose concentration in the water is, in practical terms, not dependent on the discharge.

As a test of this hypothesis, and in order to obtain a measure of the scale of the process in relation to the total sediment transported by the river, a pilot exercise was carried out using data from the 1986 and 1987 imagery derived maps contained in the JMBA Report for the reach between sections J-11-7 and J-5-6 (a distance of 76 km) and the measurements of sediment load at Bahadurabad made during the interval.

These preliminary results indicated that bank erosion in the study reach could add of the order of 15 to 30 percent to the wash load coming in from upstream, depending on the assumptions made in the analysis, but that the sand fraction is probably in balance with as much material being deposited in the form of char building/bank accretion as is yielded from bank erosion. The inference is that a large proportion of bank silt yield is transported directly to the Bay of Bengal but that the sand yield follows relatively short travel distances from bank source to either the mobile bed forms or the meta-stable chars.

The implications with regard to the consequences of bank stabilisation are substantial and so further analysis was carried out using data generated by ISPAN, under the FAP-19 study, from interpretation of the Landsat imagery listed above. The ISPAN output was in the form of land areas, defined as floodplain, char and sandbar for a series of 2 km wide transverse strips across the braid belt, for each of the images. This data was then further processed by BRTS to derive volumes of material, based on the average depth of the river and relative heights of chars and sandbars obtained from analysis of the BWDB cross-sections. The results strongly confirm the earlier hypothesis. Even allowing for some degree of uncertainty as to the silt content of the chars, the plot reproduced as Figure 6.10 shows clearly the close relationship between bank erosion and char building over a period of time. Also apparent from Figure 6.11 is the far lower correlation between bank erosion and sand-bar growth and decline which is consistent with the hypothesis that the sand-bars reflect the relatively more uniform annual transport of sand fraction sediment through the system, whereas the chars represent long-term sand, and to a lesser extent silt, storage

Figure 6.12 illustrates how the amount of bank erosion and the change in char and sandbar volume, accumulated over the total time period, has varied over the length of the river. The major island zones and inter-island nodal reaches show up very distinctly, confirming the close relationship between bank erosion and char expansion and suggesting that the sand fraction products of bank erosion do not travel far before becoming deposited in a char.

Figure 6.13 shows how the overall balance for the sand fraction only has varied over the different time periods. It is apparent that the production of sand by bank erosion was unusually high in the period 1986-88 but that this was not reflected in an equivalent increase in the total sand volume of chars and sandbars. The inference is that the balance was washed out of the system during the exceptionally high flow years of 1987 and 1988. The equivalent plot for the 1990 to 1992 period shows a compensating net increase in char and sandbar volumes, suggesting that the river is now gradually recovering its balance.

6.7 Char Development

The 1:250,000 mapping published in 1914 shows the single thread planform familiar from the Wilcox map beginning to break up into a braided pattern with clusters of islands forming in the vicinity of Bahadurabad and Sariakandi and to the south of Sirajganj. By 1956 the location of the island groups had changed to that shown on Plate 7. Of interest is the inference that a large island cluster south of Sirajganj was in the process of becoming attached to the left bank. A similar process appears to be taking place currently opposite and to the north of Sirajganj, where the two large islands referred to as D and E are expected to become attached to the left bank within a matter of a few years; the Bhuapur Channel is showing all the signs of terminal decline.

Another feature of note is that the island group referred to as C was in 1956 to the north of the Sariakandi macro bend whereas by 1973 it had transferred to the south of the bend. At the same time Island D has moved downstream to overlap and almost merge with Island E. It is a matter of conjecture as to the link between these two movements and the relative cause and effect. Island B has in contrast remained remarkably stable in location and has steadily grown from an area of about 29 km_2 in 1956 to some 8.7 km_2 in 1992. However the migration of Island C has resulted in an excessively long inter-island reach and an unusually elongated tail to Island B. The braiding pattern is abnormally confused in this reach and it is difficult to pick up a dominant waveform; indeed the river appears in some way to be attempting to insert an extra half wavelength to rectify the distortion.

The persistance of these major island groups since 1973 is well illustrated by Plate 11 from which it can be seen that the cores of all but the ill-defined Island A have been unaffected by the changes that have occurred to the outer margins.

6.8 Braiding Intensity

Preliminary inspection of 1:250,000 scale satellite images of the Brahmaputra river suggested that it could be divided into reaches with distinctive and persistent geomorphological features. On the basis of this visual inspection, seven sub-reaches were identified as shown in Figure 6.14. Braiding intensities, number of anabranches and overall braid belt width analyses were carried out by the BUET team for the years 1973, 1978, 1981, 1987 and 1989, to which the analysis for 1992 has been added, using a methodology based on a paper by Howard et al (1970), which was also used in the JMBA study. The full results are given in Annex 2.

These preliminary results confirm that reaches of relatively high and low braiding intensity (E) alternate, but with an overall tendency for braiding intensity, number of channels and overall width to decrease downstream of Sirajganj. However this is by no means a stable pattern, and the braiding index has been as high as 4.9 for Reach 6 as recently as 1987.

Each of the upper four reaches had in 1989 a significantly higher braiding intensity than in 1973, although the trend is never steady, and higher intensities have occurred in the period of record. Similar trends are also evident in the numbers of anabranch segments (N) and overall widths (C). There is in general a tendency for N and C to increase up to 1987 but then to decline somewhat in the period 1987/89. This could be part of the natural variability or it would be related to the high magnitude flood of 1988 (return period of the order of 100 years) which might have had the effect of silting in small char top, island top and flood plain channels and building attached bars at the flood plain margins to reduce both N and C. No strong relationship between the number of anabranches and their relative sizes was found (see Annex 2).

6.9 Bed Level Trends

6.9.1 Specific Gauge Analysis

A specific gauge analysis can be used to determine if there are any trends with time in the elevation of the water surface corresponding to a given discharge. In this study, the records from Bahadurabad between 1963/64 and 1988/89 were used. This is the only flow gauging station on the river in the relevant reach and this is the only place that a specific gauge analysis can be performed without invoking assumptions.

The limitations of the gauging records must be taken into account when interpreting these results (see Annex 2) but with this proviso the following inferences can be drawn.

When analysing the records to identify any trend it would be inappropriate to use least squares regression because of the high degree of noise in the data. Instead application of a robust assessment of trend and non-homogeneity based on 3-point moving medians was undertaken. The results indicate that the stage-discharge relations for all six discharges do not show any significant trend at a 5 percent confidence level.

A similar analysis was carried out for Sirajganj but in this case some assumptions had to be made because at site flow measurements were not available. The main assumption was that while the flow remained within bank there was a simple unique relationship between the flow at Bahadurabad and that at Sirajganj on the same day. Best fit relationships were then generated between the water level at Sirajganj and the flow at Bahadurabad for each of the hydrologic years, two power curves were found to give a satisfactory representation in each case. The remainder of the analysis was as for the Bahadurabad data.

The results shown in Figure 6.15 indicate that there has been no significant change in water level for a given discharge and therefore probably no sustained aggradation or degradation of the section. The small trend gradients shown are small in relation to the measurement errors involved and inclusion of data from 1955 to 1960 would actually reverse the trend in some cases. The apparent cyclical trend in water levels is of possible relevance; the amplitude of around 1 m and wavelength of around 7 years is comparable to that observed at Bahadurabad. Although not conclusive this is additional evidence that the river appears to be in dynamic equilibrium.

Stage changes like this are characteristic of a large, braided river with a highly mobile bed. Pulsed movement of bed load is widely observed in rivers. It is sometimes attributed to unsteady supply from outside the channel associated with non-fluvial events such as tectonic events. However, bed load pulses are known to develop even in cases of steady sediment supply in flume experiments (Thorne et al., 1987) and so they would probably be a feature of the Brahmaputra with or without external influences.

Any persistent trend in the stage-discharge relations over the twenty five years period of record could be indicative of net degradation or aggradation of the channel. It may, therefore, be concluded that the records do not indicate net aggradation or degradation over the whole period of record.

6.9.2 Analysis of River Cross-sections

BWDB Morphology Division annually survey cross-sections across the Brahmaputra River at fixed locations. During most years a minimum of 32 sections are surveyed and additional sections are added when a special need arises. In total approximately 1000 cross sections have been surveyed representing a substantial record of the river from 1964 to the present day. These data have been analysed by BRTS with the aim of establishing the principle characteristics of the river channel geometry and whether there have been morphological trends in the river's development over this period, both over the length of the river and with respect to time.

Cross-sections were surveyed at 2 km intervals during 1986-87 and this detailed record of the river topography has been used in the BRTS 1-D hydrodynamic model of the Brahmaputra River. It was found that the model calibrated using these sections gave a good prediction of water levels for the full 25 year period 1964 to 1989. This indicates that although the sections are continually changing the river was most probably in a state of dynamic equilibrium over this period; that is to say changes over a particular reach may be very significant but properties over the full length of the river remain relatively constant. Analysis of these cross sections contribute to the understanding of the morphological changes that have taken place during the last 25 years.

6.10 Bend Characteristics

Rapid bank erosion, that is to say rates of greater than 300 m/y, is in almost all cases related to distinctive anabranch bends. Consequently, a considerable amount of study effort has been expended on analysing the life histories of all the persistent bends that could be identified on the satellite imagery, The objective being to define some measurable characteristics that could provide the basis for the prediction of future bend development.

The first difficulty was to identify bends that were truly persistent for several years and not those that died to be replaced by a new bend that was only slightly offset from the first. The initial screening of the imagery covering the period 1973 to 1990 produced a list of 29 bends of which 10 were concave to the right bank and 12 to the left bank; the remaining seven bends were associated with mid-stream chars. Certain interesting features that emerged from this initial screening were:

(a) The low flow channel widths ranged from 375 m to 1625 m and the radii from 1,000 m to 16,000 m. There was no discernible difference between the left bank and right bank in this respect.

(b) There was no obvious variation in the number of bends active in any one year, although the sample is rather small for such trends to become apparent unless they are very pronounced.

(c) The major active bends tend to be concentrated between Fulchari and Kazipur on the right bank and opposite Sariakandi and Bhuapur on the left bank and a scattering on both banks south of Sirajganj.

(d) Of all bends analysed, only about one quarter displayed a complete life cycle, moving through steadily tightening radius until they died; it is these bends that do the most damage. Other bends were destroyed by other larger scale channel form developments before they could become fully aggressive.

(e) Only two persistent bends have been identified, that is bends that have died and then recurred at almost the same location a few years later. These are both on the right bank, one at Sariakandi and the second the embayment immediately to the north. It will be noted that both are situated on the concave side and close to the apex of the macro scale change of alignment of the braid belt.

Based on this information one could expect 12 to 15 bends to be active in any one year of which 6 would be on the right bank and 6 on the left. Since the average life span is about 4.5 years, normally only 50 percent bends would be in their peak aggressive range at any one time. The situation in 1988 through to 1990 was on these ground very unusual, with at least six major bends active on the right bank at one time. The 1991 monsoon season, with only three locations reporting severe erosion, is more normal.

A second more rigorous screening was then carried out as described in Section 3.3.1 of Annex 2. This resulted in only eight remaining for further analysis. The histories of these bends and the analyses that were carried out are described in Annex 2. As might be expected from the dynamic character of the river, relationships between radius, width and bend migration were generally weak. The plot reproduced in Figure 6.16 does nonetheless provide an outer envelope and an overall pattern from which two important facts can be inferred. The first is that bends that develop a low flow outer bank radius of less than 4,000 m are very likely to develop into aggressive eroders but that when the radius tightens to 2,000 m they enter a rapid decline; this is consistent with conventional theory linking meander bend development and decline with the ratio of radius to width, if allowance is made for the changing values of these parameters as the discharge increases towards dominant discharge. The second is that if the bend radius is less that 4,000 m erosion rates are very unlikely to exceed 300 m/y.

It was also noted that bends in an aggressive phase typically did not migrate downstream, as would be expected from meander bend theory and experience, but punched their way laterally into the bank. A good example of a bend of this type was the one that caused so much damage at Jalalpur prior to 1990. Although these bends are devastating in their behaviour they are fortunately not very common. The analysis of bank erosion shows that rates at this level have a probability of occurrence of less than 5 percent. The conclusion is that forward planning to guard against the impact of aggressive bends is probably impracticable and that when they do occur the most appropriate response will be in the form of emergency measures.

Attempts to derive relationships between non-dimensional parameters were unsuccessful and therefore the inferences drawn are only applicable to the study reach of the Brahmaputra.

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The conclusion is that the practical prediction of bend development will always be limited by a low level of confidence. Certain characteristics have been identified that warn of impending development of an aggressive bend but there is a greater than 50 percent chance that any such bend will be overtaken by other developments before it can evolve further. With further analysis as more data becomes available over time it may be possible to identify secondary influences that affect the life expectancy of a bend and thereby to improve the prediction confidence. At present the data set is too small for this to be possible.

6.11 Planform Dynamics

The major underlying questions regarding the planform of the river are:

- is the braid belt consistently migrating as a whole and if so in what direction and at what rate?

- is the river overall width increasing and if so is that an increase in water width or of island size?

- is the pattern of islands and inter-island nodal reaches changing over time?

The more immediate questions to be addressed by this study are what is the current rate and pattern of movement of the right bank of the river and what is the predicted future rate and pattern.

The elapsed time since the great avulsion that took place at the end of the 18th is too short in relation to the timescale of river development for it to be possible to predict whether the river has a sustained tendency to migrate in any particular direction and at what rate. What can be stated with confidence is that over the past 35 years the right bank of the river has consistently eroded more than the left bank and although this distinction has been less marked during the past 20 years, the rate of erosion on the right bank remains substantially higher than on the left (see Plate 5); moreover accretion on the right bank has been almost negligible while on the left bank it roughly balances erosion. This is reflected by the planform of the river: the alignment of the right bank is seen to be clearcut and relatively straight, in marked contrast to the rather ill-defined and irregular left bankline with its numerous semi-attached chars.

It is difficult to define the centreline of a braided river since the centroid of the channels is constantly shifting laterally, but by applying the simplistic definition that the centreline is the midpoint between the two outer banks it is found that this locus has over the last 35 years shifted systematically over some reaches of the river while remaining notably static in others (see Plate 10). The former are, as would be expected, associated with high rates of bank retreat; the latter include the reaches to the south of Brahmaputra and Sariakandi respectively and in the vicinity of the Jamuna Bridge site. The static lengths coincide with those parts that have earlier been identified as principal nodes of the system. It is slightly surprising that movement of the centreline between these nodes has been in all cases towards the west.

Attempts to quantify the movement of the centroid of the multiple channel from the historic cross-section data produced results that were difficult to interpret and appeared to be inconsistent with the evidence of maps and satellite imagery. It was concluded that this arose from a combination of the unrecorded shifting of the reference monuments as bank erosion took place (it was found through survey undertaken by the BRTS that the actual positions of many of the sections were several kilometres away from those shown in the official record map) together with inconsistency in the direction in which the section was surveyed (see Annex 2 for further details).

Interpretation of the satellite imagery undertaken by FAP-9 does however show clearly that there has been not only a substantial net accretion of charlands since 1973 but that this has taken place mainly on the left side of the river with a consequent shift of the centroid of the channels towards the west. This tendency is most marked in the reach between Sariakandi and Sirajganj (Islands D and E) and south of Belkuchi (Island F). It has been noted earlier that the eastern anabranch in the Island D and E reach has been steadily declining in size over this period and now carries less than 10 percent of the flow. From the both the braiding analysis and the cross-section analysis it is known that anabranches taking such a low proportion of the total flow are very unusual and the inference is that, unless it makes a sudden unexpected recovery, Island D and E will become attached to the left bank within a matter of a few years. This will result in a major westward shift of the nominal centreline and will considerably accentuate the macroform sinuosity of the river.

Closer examination of the bankline movement statistics obtained from the satellite imagery and comparison with the 1956 mapping shows that the average rates of right bank erosion over the past 19 years have been remarkably consistent with the equivalent long-term erosion rates over a 35 year period. This would strongly suggest that these rates may be expected to be continue for at least several decades. Based on this assumption the predicted bankline in the year 2011 has been computed for both 50 percent and 5 percent exceedance probability, as shown in Plate 13. The irregularity of the lines on the left bank reflect the high variability of erosion and accretion that has been noted earlier. This analysis also suggests that the river will continue to move westwards, even without the effect of the attachment of Islands D and E.

The bankline forecast shown in Figure Plate 13 provides the best estimate of how the river will develop based on historic trends. The only proviso concerns the fate of Islands D and E. If they do become subsumed into the left bank floodplain then it should be anticipated, in the absence of bank stabilisation, that the process of bank erosion and linked char growth will transfer to the new single anabranch. The consequence of this change on the rate of bank erosion is at present unpredictable.

6.12 Overall Significance of Results

The morphological study has provided essential insight into the long term trends of the river in terms of both channel geometry and planform. Although there remain some areas of uncertainty, it has provided answers to the three fundamental questions set out at the beginning of the previous section.

It appears clear that continued westward movement of the right bankline for the next several decades is a high probability unless structural stabilisation is provided. It is also probable that the rate of this movement will on average be similar to that which has taken place over the past 35 years. In the shorter term the rate will vary locally with periods of quiescence followed by periods of more active erosion; historically these quiet periods have not lasted for more than 4 to 5 years and are typically followed by erosion rates of around 200 to 300 m/y as the bankline "catches up". Higher erosion rates of up to 750 m/y (the highest recorded on the right bank) occur more rarely (less than 5 percent of the time) and their locations appear to be random. The duration of such rates is never more than 8 years and usually no more than 4 years. Despite the local devastation that they cause these aggressive bends are not therefore the principal form of bank movement. Most bank erosion is caused by shortlived bends, with radius greater than 5,000 m, that in some cases migrate downstream at up to 1 km a year but in others are washed out within one season leaving only a shallow embayment.

This understanding of the bank erosion process, the role of bends and the dominant waveforms with which they are associated, provides the essential background for the planning of stabilisation works at the study reach level and the setting of priorities for early implementation of individual elements.

7. RIVER BEHAVIOUR PREDICTION

7.1 Channel Geometry Characteristics

A distinction has to be made between the characteristics of the individual thalweg channels, which form the distinctive pattern at low flow, the combination of these channels to form the major anabranches of the anastomosed system, which are the principal planform features between dominant and bankfull discharges, and the total composite channel as a whole. Mean depth in particular will vary considerably depending on the definition.

The analysis of historical cross-sectional data has shown that there is considerable variation in all the at-a-section channel parameter values, both over time and longitudinally down the river, but that underlying this there is a strong persistent pattern. Thus while there are found to be very large changes in cross-sectional area both spatially and temporally, the relationship between stage and conveyance remains remarkably consistent over the length of the river and over the duration of the records (Figure 6.15). This is all the more notable in view of the steady increase in overall bankfull channel width that has occurred in recent decades, as shown graphically in Figure 7.1.

<u>Mean Width and Depth</u>

It was also found that the relationships between discharge and channel width and mean depth proposed by the JMB Study provided a good description of the median values for these parameters but that there was a considerable distribution of observed values about these medians. It can be seen from Figure 7.2 that the width of a single thalweg channel can range from 80 percent greater than the value predicted by the relationship to 40 percent less. Similarly Figure 7.3 shows that the mean depth distribution is more skewed with possible values ranging from 75 percent greater to 30 percent lower.

If it is accepted that the channel conveyance characteristics have remained the same, and the evidence provided by the good correlation between the water levels predicted by the 25 year runs using the 1-D hydrodynamic model and historic water levels points strongly to this inference, then the more than 35 percent increase in wetted width of channel indicated by Figure 7.1 would be expected to result in a decrease in the mean depth of the order of 17 percent. This may partly explain the scatter of values shown in Figures 7.2 and 7.3.

These relationships provide useful guides for planning purposes and, for example, provide the basis for predicting, within the confidence limits described, the channel shape that would arise if the number of thalweg channels were to be artificially reduced or if the dominant discharge were to be changed due to intervention upstream.

<u>Maximum Depth</u>

The JMB study analysed the bathymetric survey data held by BIWTA; the maximum recorded depth north of the confluence with the Ganges approximates to 36 m below the 100 year flood level, although deeper scour (40.5 m) was reported off Ranigram Groyne in August 1986, as recorded in Appendix B. Data from surveys undertaken by BIWTA to investigate bend scour were analysed, the 27 measurements giving depths equivalent to 13 to 30 m below the 100 year flood level, with a mean of approximately 20 m and a 1 in 7 chance of the depth

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exceeding 24 m. A similar analysis of the 26 measurements taken by BIWTA to investigate confluence scour indicated a mean of approximately 21 m and a 1 in 7 chance of the depth exceeding 26 m.

Analysis of the BWDB cross-section data by the BRTS produced the result shown in Figure 7.4 from which it can be seen that there is only a small probability of the depth exceeding 20 m below dominant discharge level, or 23 m below the design flood level, anywhere on the river. Moreover, there is evidence that the incidence of severe scour is higher on the left bank than on the right bank.

It has been possible to replicate these extreme depths using the 2-D morphological modelling system and to demonstrate that confluence conditions, where two thalwegs meet downstream of an island or sandbar, provide the degree of velocity amplification that is associated with such scour. It was also shown that the scour tended to adhere to one or the other bank. Depths predicted by the model and observed in the Test Area 1 surveyed by the BRTS team were both consistent with the relationship derived by the JMB study but again showed that a variation of at least 20 percent on either side of the value given by the relationship could be expected.

The analysis of cross-sectional data (Figure 7.5) indicates that the maximum channel depth may be as high as 5.5 times the mean depth although a value between 2.0 and 4.5 is more usual, and that this ratio is not significantly influenced by the relative size of the anabranch. Application of the 2-D model also suggested that there is in effect a limiting value on the ratio of about 5.5 (see Figure 3.25) associated with a velocity amplification of about 2.2. Such velocity amplification was observed in the physical models for Kazipur and Fulchari, both locations of unusually deep scour in 1990/91.

It seems therefore that a maximum scour depth of 35 m could theoretically occur due to velocity amplification arising from the confluence of two deep channels. This would however take a finite time to develop and it is unlikely that conditions in the river at present would ever be sufficiently stable for this to happen. However if the planform were to be stabilised there could be the opportunity for such depths to develop.

The location of deep scour will remain hard to predict because the conditions that cause it are usually of a transitory nature and may come about during the course of one monsoon season. The example of Kazipur shows that once the deep scour has developed it is capable of being sustained in the absence of the original conditions and may migrate downstream. For the purposes of designing stabilisation works it is prudent to assume that such scour can occur anywhere along the river, until such time that further evidence to the contrary becomes available.

7.2 Flow Velocity Variation

As explained in earlier chapters (see sections 4.6.1 et seq., and 5.3) a probabilistic approach based on mathematical and physical modelling has been used to predict the maximum velocity likely to occur at a point near the river bank and, further, likely to occur near river training structures. Full details are given in Appendix C.

7.3 Long term Bed Level

The first indications that the river within the study area is in dynamic equilibrium were provided by the results of the specific gauge analyses which showed no significant change in the water level for a given discharge (Section 6.9).

Further evidence was provided by the analysis of historical river cross-sections which showed that while there was some short-term variation in the elevation of both the locus of the centroid of the channel section and that of equal conveyance value, over a period of almost 30 years there have been no significant changes (Figures 6.15).

Although the duration of records is relatively short, it seems reasonable to infer that the river is in dynamic equilibrium or, at the least, very close to that state, despite the strong widening tendency described earlier.

7.4 Bend Development

A thorough analysis of bends selected from the satellite imagery covering the period 1973 to 1992 has failed to show any very clear picture of the pattern of their development. On the contrary it has shown that bends are in general very short-lived and the large majority are erased after a single season. Those bends that survive for several seasons have an increasing chance of becoming aggressive; it appears that as the bend radius reduces below 4,000 m there is a significantly increased probability that the bend will be associated with high rates of bank erosion (Figure 3.27). If the radius becomes less than about 2,000 m the larger channels rapidly lose their erosive tendency and will normally not survive a further season. This is supported by the 2-D modelling which showed that when the ratio of radius to width falls below a value lying between 2 and 3 the bend is unable to sustain itself and a cutoff will inevitably occur.

The primary objective of this analysis was to determine whether any consistent patterns of anabranch bend behaviour could be discerned as an aid to short-term prediction of bank erosion progression. The focus was on substantial bends that had their concave face contiguous to the bank at the time and were therefore known to be associated with severe bank erosion. For completeness, bends of similar significance eroding central chars and the left bank were also identified.

The only source of data on the planform of bends that is suitable for this purpose is the dry season Landsat imagery dating back to 1973. Bends were identified by comparing the low flow braid pattern in consecutive years and picking out bends that had a recognisable life of more than two years. There is an element of subjectivity in such an approach and this should be taken into consideration when interpreting the results. The first screening produced the following results:

(a) Over the period 1973 to 1990, with gaps in 1974, 1975, 1979, 1982, 1983 and 1989, 29 bends were identified as described earlier; of these 10 were concave to the right bank, 12 to the left bank and the remainder were contiguous to mid-stream chars. Although this is a small sample it may be inferred that there is an approximately equal distribution of bends between both banks.

(b) 6 bends concave to the right bank and 8 concave to the left bank were picked as having lives extending over at least 3 years. These represented only 60 percent and 25 percent respectively of all bends identified during the primary screening. The bend lives ranged from 3 to 7 years with a mean of 4.4 years and standard deviation of 1.2.

(c) The low flow channel widths ranged from 375 m to 1625 m and the radii from 1,000 m to 16,000 m. There was no discernible difference between the left bank and right bank in this respect.

(d) There was no obvious variation in the number of bends active in any one year, although the sample is rather small for such trends to become apparent unless they are very pronounced.

(e) The major active bends tend to be concentrated between Fulchari and Kazipur on the right bank and from opposite Sariakandi down to Bhuapur on the left bank, and a scattering on both banks south of Sirajganj.

(f) Of all bends analysed, only about one quarter displayed a complete life cycle (see Section 3.3.1) moving through steadily tightening radii until they died; it is these bends that do the most damage. Other bends were destroyed by other larger scale channel form developments before they could become fully aggressive.

(g) Only four persistent bends have been identified, that is bends that have died and then recurred at almost the same location a few years later. Two were on the right bank and two on the left; the right bank examples are the embayments to the north and south of Sariakandi; the left bank examples are north of Bahadurabad and near Tangail.

(h) Based on this information one could expect 12-15 bends to be active in any one year of which 6 would be on the right bank and 6 on the left. Since the average life span is about 4.5 years, normally only about half of the bends would be in their peak aggressive range at any one time. The situation in 1988 through to 1990 was on these grounds very unusual, with at least six major bends active on the right bank at one time. The 1991 monsoon season, with only three locations reporting severe erosion, is more normal.

Following the rectification and registration of the imagery by the FAP-19 team, a more rigorous selection of bends was made based on the following criteria as described in section 3.3.

The conclusion is that the practical prediction of bend development will always be limited by a low level of confidence. Certain characteristics have been identified that warn of impending development of an aggressive bend but there is a greater than 50 percent chance that any such bend will be overtaken by other developments before it can evolve further. With further analysis as more data becomes available over time it may be possible to identify secondary influences that affect the life expectancy of a bend and thereby to improve the prediction confidence. At present the data set is too small for this to be possible.

One characteristic that is of potential use for planning embankment alignments for relatively short life horizons is that in most cases the aggressive bends have a relatively low ratio of lateral to longitudinal movement. This means that they typically punch into the bankline rather than shave slices off it. However there are exceptions to this rule where the bend has followed the initially lateral movement by a downstream migration and actually regenerated again in a new location.

7.5 Bankline Movement

The methodology adopted for the analysis of bank movement since 1953 has been described fully in Annex 2 to this Report. It has been confirmed that the right bank has experienced net erosion over its full length since 1953 and that some reaches have suffered more than others. The distribution of the total erosion over this period is shown in Plate 3 and for two intermediate periods in Plates 4 and 5, from which it can be seen that the distribution of bank erosion has varied considerably over the total period.

It was expected that there would be some correlation between bank erosion and the position of the large islands since it is in these reaches that widening of the river has been most marked (Plate 9). Accordingly the rates of erosion were plotted on an annual basis averaged over 10 km lengths of the river with the result shown in Figure 7.6. Plots of the rates of erosion against time for each 500 m length showed remarkable consistency of form (see for example Figure 7.7 and it was concluded that the pattern of the last 20 years provided a reasonable basis on which to base a predictor of future movement. The rates for each 500 m reach of the river were grouped in sets of 20 and from each of these sets the mean erosion rate and standard deviation was calculated. The plot shown in Plate 13 was obtained by extrapolating the median and 95 percent non-exceedance values for each 10 km reach and calculating the respective banklines as they might appear in 19 years time. Lines that are close together are indicative of a very consistent erosion rate while those that wide apart indicate reaches where erosion and accretion have alternated. It is clear that the uncertainty is very much higher on the left bank than on the right.

what is the probable reason

The same data set was used to analyse the frequency distribution of different rates of erosion. For this purpose only those sections experiencing active erosion within a 500 m reach in any one year were considered. To simplify interpretation, the output was presented in the form of cumulative frequency plots for four reaches of the river (Figure 7.8): Reach 1 extending from the Teesta confluence down to the southern end of Island B; Reach 2 covering the length where the original BRE remains down to the upstream end of Island C; Reach 3 containing Islands C, D and E down to Sirajganj and Reach 4 the remainder of the study area down to the Hurasagar confluence. The distribution pattern was remarkably similar for all reaches and for all periods. Only two periods departed from the general pattern; these were 1980-84 and 1990-92 (see Figure 7.9). It can be seen that rates of erosion above 250 m/y may be considered unusual and above 500 m/y as relatively rare. The very high rates experienced at Kazipur and Jalalpur in the period 1989 to 1991 fall into this latter category and a link with the aftermath of the 1988 flood flows seems very probable. Two provisos need to be emphasised: the rates are averaged for the periods concerned, which ranged from two to four years, and may therefore underestimate peak rates; the frequencies relate to those portions of each reach that was experiencing some degree of erosion. The proportion of each reach that was being eroded during each period is shown in Table 7.1. Thus the inference is that if a section of the bankline is subject to erosion then it is possible to predict with a high level of confidence the risk of different rates of erosion occurring at any one time.

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Table 7.1 Proportion of Reach under Erosion, 1973-1992

Period	Reach 1	Reach 2	Reach 3	Reach 4
1973-1976	66.07 %	97.83 %	84.31 %	79.55 %
1976-1978	80.15 %	89.13 %	85.29 %	74.24 %
1978-1980	64.89 %	78.26 %	62.75 %	64.39 %
1980-1984	86.26 %	97.83 %	78.43 %	53.03 %
1984-1987	87.02 %	56.52 %	81.37 %	93.94 %
1987-1990	93.89 %	91.30 %	91.18 %	91.67 %
1990-1992	48.85 %	45.65 %	70.59 %	57.58 %
Mean	74.59 %	79.50 %	79.13	73.48 %

The analysis was taken one step further by investigating the duration of different levels of erosion. Again the results were very consistent for the four reaches. From the data set out in Appendix A of Annex 2 and illustrated in Figure 7.10 it can be seen that for the whole study area the average duration of all categories of erosion rate lie between 3 and 3.5 years, with the extremes (extremely rapid) being closer to the 3 years.

The pattern for extremely rapid, very slow and rapid is very similar (very rapid is distorted by perhaps one or two special cases) with only 20 percent of cases lasting more than 4 years and less than 2.5 years. Thereafter about 5 percent last between 5 and 6 years and neither extremely rapid nor very rapid ever last more than 8 years.

The normal category differs significantly only in that the 5 percent level is extended to almost 7 years, after which the curve tails off rapidly. The slow category stands out distinctly from the others with 20 percent of cases lasting more than 6 years and the 5 percent level approaching 9 years.

In short, most of the time (80 percent) any state will not last more than 5 years, or 6 in the case of slow, and the likelihood of it lasting more than 8 years is very low. There is however little difference between the categories with the notable exception of slow.

The pattern is very similar for all reaches. The slow category consistently shows the somewhat longer durations but this is particularly exaggerated in each 4 and to a lesser degree in each 3.

A similar analysis was carried out for the periods of accretion with the results shown in Figure 7.11 and 7.12. The similarity with the euivalent erosion plots is very apparent and provides a very useful guide as to the probable maximum duration of periods of accretion. This has direct relevance to the planning of bank stabilisation works at locations such as Sirajganj and Sariakandi, which are currently experiencing periods of accretion immediately upstream.

7.6 Char Evolution

Clusters of islands were a feature of the river as depicted on the Rennell map dated 1765. Immediately after the major avulsion the river adopted a primarily meandering planform but this had begun to break down by 1914 (Plate 8) and by 1953 very distinct clusters of islands had emerged again (Plate 7). The locations of these clusters are much as seen today with the notable exception of Island C, which was located north of Sariakandi.

The development of these island clusters into the substantial islands seen today has been a continuous process since 1953. Since 1973 there has been a strong tendency for all islands to grow in size and the highest rates of bank erosion have been directly linked to this.

Plate 11 shows very clearly that Islands D and E have gained in size at the expense of the left bank anabranch. If this trend continues, as it seems likely to do, these two chars may be expected to become attached to the left bank. This would leave a relatively straight reach between Sariakandi and Sirajganj, in which three secondary chars can be seen already to be taking shape.

Similarly, Island F has grown considerably during the past 20 years and the high rates of bank erosion between Belkuchi and Jalalpur are directly linked to this. Since this island has an unusually long thin shape, it is expected to continue to grow for the forseeable future, and bank erosion will accordingly continue in this reach.

Through the analysis of data supplied by FAP-19, it has been possible to demonstrate that there is a direct link between bank erosion and char building, although there is often a time lag between the two processes and erosion in one reach does not result in immediate char growth in the same reach. Thus whereas the 1988 flood flow resulted in massive bank erosion the growth of chars in response is occurring more slowly and the balance is unlikely to be recovered for several years to come. This is illustrated by Figure 6.13.

7.7 Overall Planform Changes

Since the avulsion that took place at the end of the 18th century, the river south of the Old Brahmaputra offtake has transformed from a predominantly meandering planform (Plate 15) through the intermediate form shown in Plate 8 to the distinctly braided form of today.

During the past 30 years the river has been becoming steadily wider. It has been possible to plot this trend over the last 20 years more precisely and this has shown that not only is the trend very consistent when the study area is considered as a whole but also that the rate of widening has been appreciably faster south of Sirajganj than to the north. This together with the evidence provided by the braiding index measurements points strongly to the conclusion that the river is continuing to become wider and more braided; individual thalweg channels (those seen as distinct channels at low water level), are becoming wider and it is reasonable to expect that they are also becoming shallower, although this trend tends to be obscured by the normal range of variations.

It is hard to judge how long this tendency will continue but it would be unusual for such a strong trend to change suddenly and on this basis a continuation for several decades seems likely. Plate 13 provides an indication of where the bankline could be in the Year 2011 based on the actual movements since 1973.

However there are other indicators that may signal a discontinuity in the trend, if only temporarily. If the Islands D and E do become attached to the left bank, as the evidence provide by Plate 11 strongly suggests, then this reach would quite suddenly become much narrower (as happened to the reach south of Bhuapur in the early 1950's). If the processes driving the widening are predominantly external then such a narrowing in one location might be expected to trigger a compensating widening elsewhere. If they are predominantly internal, such as the established a feed-back link between bank erosion and char growth, then the widening would probably be more focussed on the right anabranch between Kazipur and Sirajganj. Either way, the end result would be increased bank erosion on the right bank.

Attachment of Islands D and E to the left bank would also have the effect of enhancing the macro-planform meander amplitude and this could result in preference being given to the left bank anabranch south of Sirajganj. This would in turn favour the attachment of Island F to the right bank, a tendency that would be consistent with the pattern of island growth shown on Plate 11.

The question of whether the river is consistently migrating westward as a whole remains open and possibly the timescale since the avulsion is too short for any such trend to be firmly established. Certain portions of the river have certainly migrated a considerable distance westward since the avulsion but it is not possible to say whether this has been simply part of the adjustment to such a major shift or a part of a longer term trend. Any westward creep is certainly slow in relation to the rate of widening and the movements of the channel centre line associated with the attachment and creation of islands (Plate 10).

The continual accretion that is taking place at the mouth of the Old Brahmaputra, which is likely to result in its gradual decay as a major distributary (Plate 12) suggests that the likelihood of the river returning to that course is low. On the other hand the possibility of a further shift westward through the occupation of the Bangali-Ichamati channel as a consequence of a major flood event cannot be ruled out. To illustrate the point, Plate 14 shows the pattern of flow through a breach north of Kazipur during 1987, when the peak discharge in the Brahmaputra was only a little above average peak.

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8. OUTLINE OF THE MASTER PLAN

8.1. Introduction

The proposed format of the Master Plan was set out in the BRTS First Interim Report (April 1991) and subsequently formally ratified by the BWDB and the FPCO.

It was agreed that the Master Plan should be fundamentally a detailed strategy for the containment of the Brahmaputra River and the improved performance of flood control measures on the right bank flood plain, with a timeframe that is flexible enough to accommodate financing constraints. It will cover both structural and non-structural aspects and be geared to meeting both short and long-term objectives as set out in the Terms of Reference. It will be supported by an indicative draft plan showing the scope of structural works involved as perceived at this stage, and guidelines that will allow planners to respond to the situation as it develops. In this form the plan can be continuously upgraded in response to the changes in the river pattern and as more data becomes available.

8.2. Summary of the Master Plan

The principal components will be as follows:

8.2.1 Detailed description of the situation in terms of the river behaviour

The planform history of the river since 1973 when satellite coverage began is now known in considerable detail. The comprehensive processing of satellite imagery by ISPAN under the FAP-19 programme has produced an incomparable database which has provided the BRTS morphological study team with a much needed highly reliable source on which to base their quantitative analyses in support of earlier hypotheses.

The results from the analysis of this recently available data has not radically changed the overall picture but it has provided a greater level of detail and thereby much improved confidence in the tentative inferences that had been drawn.

Prior to 1973 the quality of data becomes progressively sparser and of lower quality but it is sufficient to provide a clear general picture of the progression of the river from a meandering channel soon after the late 18th century avulsion to the highly braided pattern today. There is strong circumstantial and deterministic evidence that this progression is not yet complete and that in the absence of intervention the degree of braiding is likely to become more intense, particularly south of Sirajganj.

At a more detailed level, considerable progress has been with regard to understanding the processes controlling sediment transport and the conditions that lead to deep scour. It is now possible to estimate the probability of occurrence and severity of such scour, with and without structural intervention, from a rational base.

Recently attention has focussed more on the charlands and the relationship between bank erosion and char building has been investigated both as a further aid to predicting future behaviour and in order to assess the possible consequences of bank stabilisation.

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Overall sufficient insight into the behaviour of the river has been gained for the purposes of drawing up a master plan with a satisfactory level of confidence.

8.2.2 Tentative predictions as to the future behaviour of the river;

The combination of morphological studies and mathematical and physical modelling has led to a good overall understanding of the river behaviour and this has provided a sound basis for the planning of bank stabilisation measures. What appeared at the outset to be a system with little if any discernible pattern can now be seen as a complex interaction of stochastic processes that in combination produce a high degree of variability but underlying which there is a considerable degree of consistency.

Thus, for example, it is not possible to predict with certainty exactly where and at what rate bank erosion will take place, even next year, but it is possible to assign probability values to these parameters that will be specific to particular reaches of the river. Moreover the characteristics of aggressive bends, those that are responsible for the relative uncommon but very destructive high rates of bank erosion, are now known and some degree of early warning is possible. This is a considerable improvement on the situation that prevailed two years ago and means both that flood containment facilities can be planned in a more rational manner and also that proactive measures can be taken by the authorities to minimise adverse impact.

It has already been mentioned that it is expected that the river will tend to become more braided, particularly south of Sirajganj. This tendency is closely linked to the overall widening of the river braid belt (outer bank to outer bank) and the steady growth of charlands.

However the wetted channel width at bankfull does not appear to have changed significantly except in situations where multiple channels have developed from a single channel and in general the relationship proposed in the JMB study:

$$W = 16.1\ Q_b^{0.53}$$

where W is the width and Q_b the discharge at bankfull, appears to be valid for the study reach as a whole.

Widening is thus seen as primarily a function of char growth but linked also to an increase in braiding.

Superimposed on this widening tendency there has over the past 35 years been a measurable net westward movement of the river as a whole demonstrated by the greater net erosion of the right bank compared with the left bank. Morphologically this is reflected in the very much more clearly defined right bankline and the numerous attached chars on the left bank, a feature that is virtually absent on the right bank below Fulcharighat. Although this trend has been weaker during the past 20 years than during the previous 15 years, it remains distinct and in the absence of contradictory evidence it is prudent to assume that and it will continue for the foreseeable future.

8.2.3 A set of guidelines that can be used to anticipate the behaviour of the river

The guidelines will be drawn up from interpretation of the behaviour and characteristics of the river that comprise elements (a) and (b).

Analysis of bankline movement since 1973 and extrapolation back to 1956 has provided a good basis for predicting future bankline retreat in terms of mean annual rate and duration. Plots of actual movement at 2 km intervals down the river show repetitive, though not clearly cyclical, patterns of quiescent periods followed often by above average rates of erosion. From analysis of this data it has been possible to estimate the probability of a certain erosion rate lasting for more than a certain period. Thus it is found that it is very rare for rates of more than 400 m/y to be sustained for more than two years. Conversely quiescent periods rarely last for more than 10 years. By looking at the past history of a particular reach it is thus possible, within broad confidence limits, to make some forecast of the probable bankline movement during the coming five years. For longer periods of time the average rates provide a better basis for prediction.

This type of predictive procedure will be set out in a form that can be followed and applied by anyone with a reasonable understanding of river morphology.

8.2.4 An assessment of the consequences of different levels of river training (intervention);

The philosophy underlying the river management programme being proposed in the master plan is the minimum divergence from the natural characteristics of the river. The concept is one of encouraging the river, by means of selected structural intervention, initially to become stabilised within its present boundaries. Only after this first stage has been achieved and the response of the river quantified would consideration of possible further confinement into a single channel be appropriate.

The principle forms and processes on which intervention could potentially have an impact have been addressed as follows:

Change in channel geometry

Analysis of river cross-sections surveyed regularly since 1964 show a considerable short-term temporal and spatial scatter in the values of the principle channel parameters, mean and maximum depth, width and conveyance, but a notable overall long-term consistency with no discernible trends and little longitudinal variation. It is reasonable to infer from this evidence that the channel characteristics are prone to a stochastic variability, this being a reflection of the dynamic nature of the braided channel, but that provided the dominant discharge is not significantly changed there should be no induced change in the mean channel geometry. Stabilisation of the bankline could however be expected to result in some greater regularisation of the thalweg pattern and therefore rather less variability in the channel geometry about the mean values.

Bed Aggradation/Degradation

Well known cases of bed aggradation following on from river canalisation have drawn attention to this potential hazard. This effect may result from either increased transport in the confined reach of the river, causing deposition in a lower unconfined reach, or from the planform stabilisation of a river that is in a naturally aggrading state.

Although the timescale of the available data is short in relation to that of this type of process, the specific gauge analyses and the cross-section data analysis both indicate that the river is in a state of dynamic equilibrium. This was further endorsed by the 1-D morphological modelling carried out jointly with the SWMC. On these grounds, stabilisation of the river alone should have no impact on mean bed level.

The effect of confinement of out-of-bank flows between continuous flood embankments has been investigated both through determining the impact on the dominant discharge and by means of the 1-D morphological modelling. The former showed that the impact of confinement only became significant if all left bank distributaries were closed off. The latter predicted that a confinement of the river to a maximum width of 6,000 m over its full length would have only minor impact on the average depth and bed level but that further confinement would result in increased sediment transport, resulting in a lowering of the bed level and consequent deposition in the Padma downstream until a new equilibrium was established.

The degree of stabilisation and management presently envisaged should therefore have no impact on the average river bed level.

Change in Macro Planform

It has been suggested that confining the river could encourage it to change from a predominantly braided planform to a more meandering form and that this in turn might mean that larger more aggressive and inherently less stable bends could become more common.

The concern seems to arise from the perception that the river is in an unstable transition between meandering and braided and that any intervention may push it over the line into a meandering phase. The further presumption is that this would be hazardous because the meandering river would be less stable. The arguments applying to potential change in channel geometry would seem to apply equally in this case. Stabilisation of the right bank alone will have no impact on the shape of the dominant discharge curve and therefore should have no direct impact on channel morphology overall. If it were to result in a lower braiding intensity locally due to channel planform stabilisation then this could only be advantageous from a human perspective provided that the stabilisation works are designed to prevent bends of dangerously short radius from developing.

Induced Bank Erosion and Scour

The problem of intervention at one point resulting in adverse impact elsewhere in the system is widely recognised. If accretion is encouraged in one location then this will normally be at the expense of net material loss at another location; this may take the form of bed scour or erosion of the opposite bank.

In the case of the Brahmaputra river, in which the products of bank erosion represents a high proportion of the net sediment throughput, there is the particular concern as to the consequences of arresting or reducing this erosion. It has been established that there is a close relationship between net bank erosion and net char growth and that the time interval linking the two processes is relatively short.

If it is accepted that the river is in dynamic equilibrium then by simple conservation of matter, any new material introduced from the bank that is not removed by the river in the form of wash load must be deposited in a char or temporarily stored in a sandbar. Although it is difficult to measure these two exchanges accurately, the computations that have been carried out support this thesis. In which case stabilising the bank may be expected to have the effect of stopping further char growth but should not on average cause any loss of char land.

It is also expected that as a consequence of reduced bank sediment contribution the scour adjacent to a stabilised length of bank will tend to increase and this will provide a preferential path for the flow. However it is known from examples such as Sailabari Groyne that this in itself is not sufficient to hold the thalweg in a position that is inconsistent with the current dominant main waveform of the river.

Looking for precedents, the only example of long-term stabilisation on the Brahmaputra is that associated with the town of Sirajganj, which dates back at least 20 years; the reach of the river immediately downstream is noted for its remarkable stability over the same timescale.

In short, the impact of right bank stabilisation works is expected to be a general stabilisation of the planform but that this may take several years to become discernible against the normal background of local planform variability.

Effect on Populated Chars

The more stable chars, whose tops are at much the same level as the river floodplain, are distinguished by their strong vegetative growth and semi-permanent population. Although the cores of many of these islands have a long life the peripheries undergo constant, and sometimes dramatic, change in shape. Stabilisation of the main river banks alone will have no affect on the dominant discharge nor the frequency and duration of inundation of the chars; the influence on the stability of the islands arising from other processes such as the shifting of thalwegs, is hard to predict but there is no known reason to suppose that the situation will become any worse than at present.

The construction of a continuous left bank flood embankment would result in some short-term increase in the frequency and duration of inundation of the chars, as would the construction of the Jamuna Bridge. The strip of floodplain between river bank and flood embankment would be similarly affected. The evidence provided by the development of new chars, with tops at floodplain level, over a period of only a few years suggests that this adjustment process could be over a similar timescale.

8.2.5 An assessment of the sociological and environmental impacts of river training and non-intervention;

Sociological

Socioeconomic field investigations have confirmed that for the riparian population perceived priorities are firstly bank stabilisation and secondly protection from major river flooding. The

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former to provide security at the most basic level and the latter to provide a major improvement to the opportunities for income generation at all levels in society. Opposition to flood protection is negligible.

Since even an intensive bank stabilisation programme would involve large scale investment spread over several decades, the problems associated with bank erosion and maintenance of an effective flood protection structure have to be addressed in the short and medium term.

The principal issues then become firstly the setting of priorities for bank stabilisation and secondly ensuring that the interim measure of flood embankment realignment is carried out in the most efficient and least socially disruptive manner and thirdly, and perhaps most critical, how to accommodate the large number of people who will become displaced through bank erosion.

It is estimated that at present about 100,000 people are squatting, illegally, on the BRE and a further 30,000 have found temporary refuge in and around Sirajganj. It is further estimated that during the coming five years alone more than 7,000 ha will be lost to erosion and at least 70,000 more people will become homeless. Since the BRE is rapidly becoming fully utilised as a refuge this will mean that a large number of people will find themselves with no immediate solution to their dilemma. The social and political ramifications are self evident.

Compared with this major problem, the issues relating to the optimal management of planned flood embankment realignment may appear parochial but they are of importance not only to those immediately affected but also to the large number of farmers in the floodplain who suffer whenever there is a breach in the embankment. At present the process of realignment is carried out with only a low level of consultation and opportunities for local participation are not exploited. Certain key issues need to be addressed. Firstly, at the local level: setting up an effective procedure for reaching agreement on the optimal tradeoff between the two conflicting requirements of providing the maximum number of people with flood protection (minimising set-back) and the social upheaval, disruption and decision that is associated with frequent realignments. This can only be effectively addressed at the local level. Secondly, at the national level: upgrading the institutional structure to enable the sensitive subject of land acquisition to be implemented in a manner that is equitable and efficient. Both these issues fall within the scope of FAP-15. Thirdly, also at the national and regional level, institutional arrangements have to be strengthened in order to provide the framework for the setting of priorities for bank stabilisation, providing the necessary financial and logistical support for planned BRE alignment, addressing the plight of the massive displaced population, and mitigating the harsh conditions faced by those left exposed on the riverside of the embankment.

Much can be done to improve conditions for this latter group in the way of relatively low cost measures that fall under the general heading of Flood Proofing: improved raised earthfill platforms for houses and commercial and public premises; road and footpath embankments; livestock and human refuge highpoints; secure dry storage facilities; reliable all season water supply; effective relief and rehabilitation services; flood warning.

The impact of flood embankment realignment can be alleviated by local consultation and greater involvement of those affected. Many of the reported problems relating to holding fragmentation and exacerbated social division are potentially capable of mitigation given the appropriate approach to the issue. Loss of productive land to borrow areas can be mitigated by a greater emphasis on sourcing of material.

For those displaced by erosion the problems are more difficult to resolve but priority should be given to formalising the occupation of the flood embankments and designing the realigned sections in order to provide the maximum refuge potential. Participation in maintenance, and possibly also realignment, by those resident on the embankment is a possibility that deserves serious consideration. In the longterm, bank stabilisation appears to offer the only sustainable solution.

Environmental

A thorough analysis of all potential physical, biological, chemical and human potential impacts arising from bank stabilisation and embankment realignment has been carried out (BRTS Environmental Impact Assessment for the Phase 1 Priority Works) and there can be no doubt that the social issues dominate by a wide margin. Negative impacts from bank stabilisation measures are all, with the possible exception of the sourcing of burnt clay bricks, related to the construction period and are both temporary and responsive to standard mitigation measures incorporated in contemporary international forms of civil engineering contract. The potential negative impact arising from the large scale use of bricks has been given particular consideration and special provisions incorporated in the contract conditions to eliminate the possibility of encouraging the over-exploitation of the limited national timber resources.

The intermediate measure of flood embankment realignment, in order to maintain what is clearly perceived by all but a very few as an important regional asset, has no identified negative impact beyond those already described under the title of social considerations. The positive aspects are substantial and well quantified: apart from sustaining the present level of agricultural production, which over a large area is dependent on exclusion of flooding from the Brahmaputra, minimisation of the risk of breach occurrence is strongly linked to disbenefits such as land degradation, property destruction, livestock loss, not to mention widespread human misery.

8.2.6 Design guides for specific river training measures

A detailed design and construction management report has already been prepared for the bank stabilisation works proposed for Sirajganj and Sariakandi/Mathurapara. This covers the full range of design conditions that may be encountered in the study reach and therefore constitutes the core of a general design guide. This core is currently being developed into the master plan design guide.

8.2.7 Costed examples of typical river training works that can be used to build up estimated costs of alternative actions

The detailed cost estimates prepared for the Phase 1 priority works cover all the elements of which bank revetment, groynes and hard-points will be composed. These will be presented in the form of costed modules that can be combined to form a wide range of structural measures.

8.2.8 Preliminary costed implementation schedules for one or more strategies;

As a result of both technical and economic assessment some of the alternatives considered in the early stages of the master plan study were eliminated. The alternatives now being costed for inclusion in the master plan are based on the concept of five levels of intervention ranging from right bank stabilisation only through to full containment of the river into a single channel. These more comprehensive options are strictly outside the scope of the BRTS Terms of Reference but it is not possible to divorce right bank stabilisation from management of the river as a whole and they have been included in outline in order to place the right bank works in context and to demonstrate how short-term works may be planned to be consistent with a longer term strategy. The main emphasis is on the scheduling of the right bank stabilisation that would form the first level of intervention, taking into account both the institutional and financing constraints, the morphological and engineering considerations, and the sociological and environmental impact.

8.2.9 A review of the organisational aspects of implementation, including operation and maintenance requirements;

A report covering in detail the planning, management, operation and maintenance of Phase 1A of the priority works has already been completed. This will be expanded to provide an overview of the longer term requirements. In essence, it is envisaged that a Monitoring and Maintenance Unit will be set up within the BWDB and equipped with all necessary survey and monitoring equipment, and land-based and waterborne plant. The Unit would become fully effective after a period of training by the construction contractor and the consultant, and expand its activities as implementation proceeds.

8.2.10 Economic assessment of the principal alternative strategies;

The economic assessment of six top priority locations distributed over the full length of the study reach has provided the basic data required for the evaluation of alternative longer term river management strategies. In parallel with that, the methodology for determining the optimal strategy for interim strategic embankment realignment is being refined using the results of the morphological data analysis that has recently become available. It is now feasible to attach probabilities to various scenarios and thereby carry out a more rational comparison of alternatives.

In general, it has been found that bank stabilisation for specific objectives such as securing Sirajganj and preventing the breakthrough into the Bangali are viable in conventional economic terms and have been recommended for early implementation. A second follow-up phase of bank stabilisation measures has been included in the economic assessment as it is anticipated that these supplementary works will be required within about five years from completion of the first phase. The timing of further stabilisation in the short-term will be strongly influenced by the response of the river to this first set of measures but a tentative schedule has been set out that could lead on to a longer term programme for the full stabilisation of the right bank.

At this time, evaluation of the longer term works fall outside the scope of conventional economic assessment. The management of the river has to be considered as a permanent national commitment not simply an investment for short-term financial returns. Bank stabilisation works are part of this longterm management strategy and become a national asset that is handed on to future generations. As such the basis of economic evaluation has to be different and the decision on whether to proceed and at what pace, becomes largely a matter of government policy relating to national priorities and the allocation of resources. In this context the stabilisation of the right bank alone emerges as an investment that will provide levels of benefit that are compatible with comparable national level development plans.

8.2.11 Status of the BRE

Planform

The position of the BRE relative to the river bank, as in March 1992, is shown in Plate 6. Some 30 km are within 300 m of the bankline and since these sections coincide approximately with reaches exhibiting the fastest erosion, it is expected that a large proportion of this length will have to be retired within the next three to five years.

The number of retirements to date generally ranges from one to three. In only two locations has it been necessary to make four retirements and the case of Jalalpur with its seven retirements is unique (see Figure 8.1).

Breaches

It has proved difficult to build up a reliable history of breaches. Most people's recollection becomes hazy prior to the 1988 breaches and official records, other than the bankline sketch maps maintained by the BWDB since about 1980, are not readily available.

From the latter it is possible to estimate that the first retirements took place, possibly in the vicinity of Kazipur around 1975, only a little over 10 years after the original construction, but that the problem started to become significant in the early 1980's. It is clear, however, that the problem has been becoming more serious over the past five years as progressively more of the BRE becomes within range of aggressive bend erosion.

Present Condition

A detailed assessment of the BRE was carried out by BRTS engineers between May and August 1991, and is described in detail in the Second Interim Report.

More than 40 percent of the length of the embankment has been occupied by temporary housing, but only part of this on both sides, and mostly situated on benches cut about half way down the slope. Over 20 percent of the length of this benching is present on both faces of the embankment and in addition, more surprisingly, over more than 10 percent of the length it is on the river side only.

It has been judged that more than 50 percent of the length of the BRE slopes on each face is in fair to good condition and requires no urgent slope face maintenance, a further 30 percent is affected by temporary housing and rather less than 20 percent requires early attention. The pattern is very similar on both faces of the embankment.

No signs of instability associated with benching have been noted, nor have any reports been received to this effect. Since this feature has been present for several years, including the 1988 high flood year, the inference is that it is not intrinsically detrimental from a practical performance viewpoint, except where any local collapse of over-steep slopes above the bench may restrict access along the BRE.

Significant damage to the river side slope due to wave action is surprisingly rare, even where the embankment is very close to the river bank. The probable explanation lies in the shallow depth of water over the bank even at high flood levels and the relatively dense low vegetation that rapidly attenuates the waves. The indications are that a distance of no more than 100 m with the right type of vegetation cover can be sufficient to effectively suppress wave action to a level that good grass cover on the embankment can withstand. There are two short lengths totalling 6 km towards the southern end of the BRE where wave damage has occurred, although this is thought to be associated with over steep slopes and poor consolidation in the hastily retired embankments.

Sociological Considerations

Attention has been drawn by other FAP studies to the potential benefit that can arise from encouraging planned occupation of flood embankments. The residents then have a vested interest in cooperating in maintaining the integrity of the embankment, and it has been proposed that future construction of the BRE should incorporate a berm which would facilitate planned occupation and avert the drawbacks of uncontrolled benching. It has been noted on the BRE that those reaches that are occupied tend to have good vegetative cover on the river side that provides protection against both rain runoff damage and wave action. Simple split bamboo revetment is often erected locally where toe erosion due to wave fretting is a potential problem.

Cross Drainage

Drainage across the BRE is considered to be significant only over a length of about 20 to 25 km, with the Ghagot River forming the approximate southern limit and Teesta the northern limit. In this stretch the general land slope is towards the Brahmaputra and the embankment can therefore restrict drainage. The problem however does not seem to be particularly serious except for pockets of lowlying land. The situation is exacerbated by the occasional breaching of the flood embankment on the southern bank of the Teesta in the vicinity of Belka.

Cross flow control structures were incorporated over the full length of the original BRE, mostly in order to provide the facility for releasing water from the Brahmaputra into what were originally distributary channels, for irrigation and fisheries interests. These channels have in the main become silted up, presumably because of insufficient incentive for the proper operation of the regulators, and their function is consequently in many cases now ill-defined.

Of the 31 existing structures, 26 are one gated, 3 are two gated. The major structures are the 12 gated Manas Regulator on Ghagot River, which is currently threatened by bank erosion, and a narrower 12 gated structure north of Kamarjani. Some 20 of the smaller structures have been rebuilt on the retired embankments.

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Maintenance

It has been noted earlier that more than half of the length of the BRE is judged to be in fair to good condition as far as routine maintenance is concerned. Less than 20 percent of both faces which are not occupied by temporary housing are considered to be in immediate need of superficial repair work, mainly consisting of repair of rain runoff damage and making up of the section where roads and footpaths cross.

8.2.12 Agro-Economic Benefits

Agro-economic benefits stem largely from the disbenefits arising from continuing river bank erosion and consequent breaching of the BRE. The latter will continue because even if the nominal "5 year" set back rule is correctly applied it will not keep the embankment out of range of the occasional very aggressive bend that can erode the bank at more than 800 m per year.

The damage resulting from a breach will vary with its location and the water levels in the Brahmaputra at the time, but it has been estimated that on average immediate agricultural damage from a single breach amounts to about Tk 10 million in the vicinity and can be as high as Tk 200 million although the average is between Tk 30 and 60 million further downstream due to extended inundation periods. Direct damage to roads and other infrastructure and the interruption to land communication are difficult to quantify but are certainly not insignificant.

If the average retirement length is 5 km and the length of the river under serious erosion threat is 100 km, and the 5 year retirement rule is applied, then on average over a period of time about four retirements will take place per year. This is consistent with the analysis of significantly scouring bends which have occurred over the last 20 years which suggests that on average six right bank concave significant bends can be expected to be discernible each year, of which about half will be in their active period. In some years this may double.

The significance of this is that unless a very well organised system can be set up for identifying potentially aggressive bends and instituting preemptive retirement of the embankment, an average of one or two breaches a year would not be exceptional. Since the breach flows on the whole enter the Bangali-Ichamati-Karatoya river system, this means that areas particularly towards the lower end experience very frequent increased inundation. This will have a permanent depressing affect on agricultural production in these areas.

8.2.13 River Transport Considerations

The proposed river bank stabilisation measures constitute a passive form of intervention and as such their effect on navigation and river traffic should, beyond minor disruption during the construction period, be negligible. County boat steps are planned at all hard points, and will therefore improve access between river and bank in some areas. At Sirajganj, a passenger ferryghat is planned on the new revetment, at a location close to that used at present.

The impact on the railway ferryghat at Fulchari and Sirajganj, and on the vehicle ferryghat, is expected to be relatively small although beneficial. These services are regularly relocated on an eroding (or accreting) bankline with the minimum of disruption, although stabilisation

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of the river bank would evidently be advantageous in the long term in reducing or eliminating the necessity to move. At Fulchari it is estimated that in five to ten years the marshalling yard will be at risk from erosion, and later possibly the main line itself, which would then require relocation.

In general, it is not expected that the implementation of river bank protection works will have any significant impact on river transport.

8.2.14 Description of Long-Term Works

The basic governing principle is to conform as far as possible to natural river planform characteristics. The objective being to maintain as closely as possible the water and sediment conveyance relationships, thereby minimising the risk of adverse consequences e.g. bed aggradation or knock-on effects. Key considerations in this respect are dominant and bankfull discharges, the former having a major influence on channel form and the latter on char elevation and the frequency-depth-duration of inundation.

A relationship between channel width and bankfull discharge has been proposed by the JMB study team. This has been checked and found to give a reasonable median value when compared with output from the cross-sectional analysis (see Annex 2). Applying this relationship to derive channel widths for both single and double anabranches resulted in a very close match with the actual channel widths interpreted from the satellite imagery. In general the sinuosity of the anabranches, as distinct from the thalweg channels, has been retained. Wavelengths have been picked out from satellite imagery as being persistent.

<u>Option 1</u> (See Plate 17)

The basic philosophy behind this approach is the stabilisation of the main features of the planform as seen in 1992 with due consideration given to the manner in which these have developed and may be expected to change with time. The method of stabilisation is based on the concept of bend stabilisation in which the fixing of the outer, concave, faces of the dominant waveform is the priority. Experience shows that if the concave face can be controlled then the natural meandering tendency of the river will look after the remainder. It is expected that the anabranches (as distinct from the individual thalwegs or braid channels) will respond in the same way.

The stabilised planform was selected starting at the Ganges confluence and working upstream mainly because this lower reach has a very much clearer waveform than the part north of Sirajganj. For Option 1, the approach was to encourage the formation of a single meandering anabranch in this reach. This does not imply necessarily a single thalweg channel but with the planform stabilised it is anticipated that this will tend to be the normal pattern, as it is at present. There will be no chars in the sense of islands exposed when the discharge is less than bankfull but there will be large areas of high sandbar. The large islands such as F and G would be induced to become attached to the main flood plain. The upstream limit of this lower reach is the southern end of Island E. From this section up to the upstream end of Island B a different approach is required to take account of the higher braiding level and the strong influence of the major islands. The starting point was the identification of the cross-overs between the islands, which are alternatively referred to as nodal reaches. A crossover is characterised by the predominance of a single channel and this effectively fixes the inflexion point of the waveform for each anabranch. With two such fixed points there are only a limited number of half wavelengths that will fit in between them. The half wavelength chosen was one that provides the best fit with the secondary island pattern. A cross check against all the available images confirmed that the chosen wavelength was appropriate (see also Plate No. 12). It was noted however that there are years in which a longer wavelength is predominant in the right hand anabranch between Kazipur and Sirajganj. It is this that is associated with the embayment in Island E.

The most difficult section of the river proved to be that between Island C and Island B. It seems that the river itself has difficulty establishing a persistent pattern in this reach, which is distorted by the macroplanform bend with its apex at Sariakandi. The long downstream tail to Island B and the ill-defined and fragmented form of Island C reflect this. It will probably be necessary to go beyond bend stabilisation in this reach and establish a firm crossover or node in this vicinity in order to induce the river to adopt a regular pattern. Two possible locations for this nodal stabilisation are indicated. The selection will depend on the planform development in this reach.

Island B itself is well defined and its main length corresponds satisfactorily to one of the river's characteristic wavelengths and sinuosity.

From here upto the Teesta confluence the situation becomes again less clear, although not as confused as the inter island b-c reach. For this section no attempt has been made to create mid-stream islands, although this does remain a possible alternative. The reason is that it would probably require more than bend stabilisation alone to sustain the pattern. Rather, the channel is encouraged to form a single meandering alignment with rather more pronounced sinuosity than with the section south of Sirajganj to compensate for the reduced braiding intensity.

Option 2 (See Plate 18)

This represents a higher level of intervention than Option 1, the main focus being on establishing a single meandering channel within which some multiple thalweg formation could be accommodated as is the existing situation south of Sirajganj.

It can be seen that the wavelength is almost uniform over the full length of the river, although the sinuosity varies (it may be necessary to increase the sinuosity in the reach between Kazipur and Sirajganj). The difficulties encountered with Option 1 in the reach between Islands B and D have been eliminated through this more rigorous imposition of a common waveform. It is significant that the overall pattern differs relatively little from Option 1, suggesting that this does represent a fundamentally "natural" configuration of which Option 2 is a modified form designed to match more closely the current planform distortions.

The broken lines indicate the outline of a simplified planform based on the current situation including the islands. This represents an alternative approach to the Option 1 concept. The differences lie basically upstream of Island D.

Hard Point Selection

In general hard points have been located at the two extremities of the length of bankline to be stabilised, these sites being just downstream of the inflexion points. Additional points have been added where the planform is less well defined and some further training of the river is indicated, or where the length to be stabilised exceeds the typical naturally occurring embayment length in that reach.

8.2.15 Description of the Short Term Works

Planning Approach

The overall strategy provides for early implementation of river bank stabilisation in critical areas. The phrase "short term" refers to works that are to be implemented on a priority basis, as opposed to the longer term programme of full stabilisation; it is not intended in any way to imply "temporary".

Early in the BRTS programme, a ranking of priority locations was carried out. An extensive programme of field visits was undertaken by BRTS team members supported by BWDB field officers. The data obtained was examined by adopting an objective multi-criteria approach based on the following principal considerations;

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

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

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

Further criteria to be considered included the importance of such issues as the possibility of the Brahmaputra breaking through to the Bangali River in the vicinity of Sariakandi, and the security of ferryghats.

Ten locations emerged from the initial ranking, from which six were selected for detailed design, namely (in alphabetical order):

Betil
Fulcharighat
Kazipur
Mathurapra
Sariakandi
Sirajganj

A full economic and sociological assessment of the six locations showed very clearly that, in conventional economic evaluation terms, investment priorities for bank stabilisation were: Sirajganj, an important regional centre where the present protection works have detiorated and which would be seriously at risk in the event of a major flood, and the reach of bankline between Sariakandi and Mathurapara, where serious bank erosion is making a breakthrough into the Bangali an increasingly likely event. While investment at these locations showed respectable internal rates of return, the financial and economic returns to bank protection works at the other three locations was poor (see Figures 8.2 and 8.3).

It was consequently agreed between the Government and IDA (World Bank) that the Phase 1 Priority Works should comprise bank protection works at Sirajganj and Sariakandi/Mathurapara. In the context of the Master Plan, these have been described as the Phase 1A works. During the first phase of the Master Plan, it is expected that hard points to stabilise the reaches immediately north of Sirajganj and immediately north of Sariakandi will be required. Accordingly, two locations north of Sirajganj - at Simla and Sailabari Groyne and two locations north of Sariakandi - one east of Naodabaga and one approximately 3 km north of Kalitola Groyne, have been included as Phase 1B. Phase 1C will comprise the remainder of the six locations selected originally, namely Betil, Fulcharighat and Kazipur (see Figures 8.4 to 8.8).

Structural Measures

The Phase 1 Priority Works are aimed at the stabilisation of the bankline in the immediate vicinity of Sirajganj and along the 15 km reach centred on Sariakandi where there is a high probability of an early breakthrough of the Brahmaputra into the Bangali river with consequent widespread disruption and loss of the BRE. These two situations are distinct in character and consequently involve different conceptual approaches to the design of bank protection measures.

At Sirajganj, more than 2 km of new revetment will be constructed, incorporating the existing Ranigram Groyne and reclamation of land between the groyne and the town (see Figure 8.2). At Sariakandi an Mathurapara (Figure 8.3) the works will consist of reinforcement of the existing Kalitola Groyne and the construction of two hardpoints, consisting of conventional bank revetment with upstream and downstream terminations, and a lightly protected cross-bar on the flood plain linking with the set back flood embankment to prevent out-of-bank flood flows from bypassing and outflanking the hard point.

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Linkage with Long Term Strategy

From the point of view of stabilisation of the right bank of the Brahmaputra and protection of the BRE, the short term measures essentially form the first part of the longer term strategy. The short term protection measures fit into the regional level pattern of hardpoints, while at the same time providing early protection to the more local interests of infrastructure, population and land.

8.2.16 Engineering Cost Estimates

The methodology used in preparing the cost estimates follows closely that described in Annex 5 of the Second Interim Report. Cost data was obtained and updated mainly during the period March to May 1992, and the estimates should therefore be considered as being at April 1992 prices.

Bill 1 items have been calculated using experience from other contracts, local estimates of cost/unit area for housing and offices, local prices for supply of equipment, etc. The Bill 1 totals approximate to 35% of the value of the measured work items, which is a fairly usual proportion.

The major unit rates have been derived from first principles, based on likely construction methods and rates of production, materials costs obtained from manufacturers for imported materials such as geotextiles, with additions for storage, transport, duties and taxes. The cost of locally purchased materials have been obtained from local suppliers or derived from earlier BWDB contracts. 25% has been added to the direct costs of measured work to allow for contractor's overheads and profit.

Customs duty and taxes have been allowed for in the rates, although it has been assumed that contractor's equipment will be brought in duty free and re-exported. The question of whether duties and taxes on materials to be incorporated in the Permanent Works will be payable by the contractor, or whether they will be reimbursed or waived, is still open to discussion. This question does not, however, influence the economic values obtained which are calculated net of taxes.

8.2.17 Construction Management

The works to be constructed on the right bank of the Brahmaputra will be of a scale and complexity not previously experienced in Bangladesh. Although upgrading and realignment of the BRE will be carried out by local contractors working on contracts won by local competitive bidding (LCB), it will be necessary to call upon international expertise to undertake the river bank protection works.

A process of international competitive bidding (ICB) will therefore be required, amongst prequalified contractors, to ensure satisfactory execution of the works. The FIDIC conditions of contract have been used in the tender documents prepared for the Priority Works contracts, and it is expected that future contracts will be prepared on similar lines. The conditions of contract provide for the works to be constructed by the Contractor (the successful bidder) for the Employer (the BWDB) under the supervision of the Engineer, usually a firm of consulting engineers of international repute who have been involved in the design process. The Engineer

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may delegate duties and authorities vested in him to his representative on site, the Resident Engineer.

Responsibilities of the Contractor

The responsibilities of the Contractor, the Employer and the Engineer are described in Chapter 5 of the Design and Construction Management Report. In brief, the contractor will be responsible for the satisfactory execution of the works including the provision of all necessary labour, equipment and materials. Particular aspects include:

- o Quality control - the quality of his own work and ensuring that it meets the requirements of the Specification.

- o Land acquisition for temporary works

- o Site accommodation and facilities - including those of the Engineer

- o Temporary Works - the design, execution and removal on completion, of all temporary works.

- o Borrow areas - the satisfactory working of borrow areas to provide materials to the required standard, and reinstatement upon completion.

- o Timely completion - provision of a programme acceptable to the Engineer for the execution of the works, and completing the works within the period allowed by the contract.

- o Defects liability - during the Defects Liability Period, the 12 months after the works are taken over by the Employer, the contractor is responsible for remedying any defects arising from poor workmanship or materials.

- o Training BWDB staff - in the operation and maintenance of the works (see section 8.2.9), particularly with regard to the Priority Works contracts before BWDB expertise is built up.

Responsibilities of the Employer

The Employer, for whom the works will be constructed, will be responsible for:

- o Giving the Contractor possession of site, and such access as is required under the contract, so that he may commence and proceed with the execution of the works. This means that the Employer must, beforehand, be in legal possession of the land concerned, and this will involve on some sites making proper arrangements for resettlement of inhabitants. Any delay in giving possession would have serious programme and financial repercussions if construction windows were missed.

- o Making timely payments to the Contractor in respect of work measured and certified by the Engineer.

- Facilitating arrangements for the Contractor to obtain visas, work permits and duty free importation of equipment.

- Provision of personnel for training - especially during the Priority Works contracts and before BWDB's Monitoring and Maintenance Unit is set up and has gained experience.

Responsibilities of the Engineer

The Engineer's responsibilities will fall broadly into two categories, those which involve direct support to and liaison with the Employer on contractual and technical matters, customarily referred to as supervision-in-chief, and direct supervision of the Contractor's activities in site.

The former will include:

- Specifying and assessing surveys to be undertaken between award and construction in the river.

- Revising and updating designs as necessary to meet the latest river morphology and bathymetry.

- Responding to any queries on design and material specification issues.

- Preparing Variation and Provisional Sum Orders.

- Foreseeing potential programme and technical difficulties and advising the Employer accordingly.

- Advising the Employer on contractual issues.

- Carrying out Quality Assurance audits

- Certification.

On site supervision will include:

- Monitoring the Contractor's activities including compliance with Conditions of Contract, Specification, provisions for environmental impact mitigation during construction, Quality Plan, and supervision of surveys and materials testing.

- Measuring the Works and checking the Contractor's submissions for interim payments

- Monitoring progress and anticipating potential sources of delay.

- Checking the Contractor's setting out, preparation of foundations, and the like

- Checking the Contractor's method statements and temporary works proposals, and referring these to the Engineer as necessary.

o Preparing record drawings

o Responding to emergency situations as they arise.

8.2.18 Finance Arrangements

Availability of finance will be one of the principal constraints of implementation of river bank protection works under the Master Plan. Stabilisation measures for critical locations such as Sirajganj and Sariakandi/Mathurapara, which are justifiable in conventional economic terms and constitute the Priority Works contracts, are likely to receive adequate support from international aid agencies including the IDA. Funding arrangements, however, for subsequent locations, where the economic justification is not so strong, have yet to be established. Given the scale of the programme, the Government of Bangladesh would clearly be unable to finance it from domestic resources.

The principal sources of finance for a major capital investment programme would include:

- multilateral funding agencies (e.g. IDA, EDF and ADB) through loans, credits and grants;

- bilateral aid agencies (e.g. KfW, ODA, etc) through loans, grants and tied export credits;

- domestic sources, such as GOB, local banks and cost recovery from beneficiaries.

Further, arrangements will need to be made for the financing of Operation and Maintenance of the works, which can be assessed as the equivalent of around 2% of the capital cost annually. The expenditure will, of course, increase with the number of bank protection works completed and taken over. It will almost certainly be beyond the scope of local cost recovery to finance this order of expenditure, and central funding will be required. The establishment by BWDB of a specialised unit to monitor and maintain the works, and its successful operation in this field may, in itself, attract donor finance for the implementation of further works.

8.2.19 Recommendations as to Follow-up Action

Just as river surveys, mathematical and physical modelling, morphological and geotechnical studies, and economic, sociological and environmental assessments have been essential to this study, to the formulation of the Master Plan and to the design of the short term works, so will such activities play a vital part in the planning and implementation of later stages of the stabilisation of the Brahmaputra right bank.

Certain activities may be mentioned in particular:

<u>River Surveys</u>

The ongoing execution of river surveys to monitor morphological changes including scour development, acquire further flow data to assist in refinement of the design of river training

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structures, and in the future to monitor the performance and influence of completed structures, is considered to be of major importance.

<u>Satellite Imagery</u>

Satellite imagery and the processing thereof by FAP-19 has proved to be an invaluable tool in improving understanding of the river's past behaviour and in predicting how it may behave in the future. Continued monitoring of satellite imagery, as it becomes available, will add to the knowledge obtained hitherto, and will help to give advance warning of high erosion attack and thereby assist in planning the sequence of construction of river bank protection measures.

In addition to the landsat imagery processed by FAP-19, and the high resolution SPOT imagery, it is understood that imagery from the ERS1 SAR Satellite will shortly be available. This radar imagery will permit observation of the Brahmaputra during the monsoon season, where cloud cover usually precludes the use of other imagery. This opportunity would evidently add further to the understanding of the river's behaviour.

<u>Operation and Maintenance</u>

It is recommended elsewhere (see the report "Provisions for Operation and Maintenance", June 1992) that a Monitoring and Maintenance Unit be set up within the BWDB. This unit would, after an initial period of training and acquisition of expertise, be responsible for the monitoring of completed structures and their regular and emergency (if the case arises) maintenance. These responsibilities would include river bankline monitoring and assessing any effects of completed structures on the bankline, and would be developed to include monitoring of the river planform using satellite imagery.

In the long term, therefore, the Unit would be able take over the two activities detailed above. However, adequate arrangements should still be made for their continuation in the shorter term.

<u>Sociological Considerations</u>

Provision will need to be made for persons displaced as river bank erosion continues, and as the BRE is retired. More specifically, arrangements will need to be made for people displaced by the construction of river bank protection works. Draft Resettlement Plans have already been prepared for the Priority Works contracts, and these include the recommendation of the early establishment of a Monitoring and Resettlement Section within the Bangladesh Rural Development Board to keep those affected informed of developments, to monitor the moving of those persons, and to ensure that any problems which arise at that time or subsequently are dealt with promptly. Similar arrangements will need to be made as more works are constructed in the future.

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9. SUMMARY OF DESIGN CONSIDERATIONS

9.1 Design Objectives

The objectives of the river bank protection works are to stabilise the right bank of the Brahmaputra in specific locations where bank retreat is threatening the integrity of the Brahmaputra Right Embankment or where important population centres and infrastructure are being threatened and, in the long term, to stabilise the entire right bank from the Teesta river in the north to the Hurasagar in the south, thereby providing permanent protection to the BRE.

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

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

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

Full details relating to the design of the Priority Works are given in the Design and Construction Management Report.

9.2 Hydrological Criteria

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

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

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

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50 percent probability that work can be carried out in the dry over a continuous period of 160 days.

9.3 Hydrodynamic Criteria

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

Based on the results of a number of physical model tests and interpretation of the 1-D and 2-D mathematical model results it has been shown that there is a relationship between the mean velocity in the river section as a whole and the maximum velocity that can develop near the bank and around artificial projections such as groynes. The derivation of these values is described in Appendix C and the results have been quoted earlier in Table 4.6.

The formula used previously by the JMBA has been used to determine armour size, and the results compared with those derived from wave considerations (see Section 9.4). Only in the case of straight revetment did wave action prove the governing criteria. At upstream terminations and groyne heads, the higher flow velocities because the critical factor for armour sizing.

9.4 Wind wave action

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

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

The screening of the data disclosed a number of errors which were corrected by the Meteorological Department. The possibility of further less obvious errors cannot be excluded. A reasonable consistency was found between the five sets of data and from analysis of the data the design significant wave height was found to be 1.0 m with period 3.0 seconds.

9.5 Geotechnical Consideration

9.5.1 Soil Characteristics

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

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

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

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

The vertical stratification for design of the Priority Works was assumed to be as shown in Table 9.1.

Table 9.1 Soil Stratification

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

9.5.2 Liquefaction

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

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

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

9.5.3 Piping and Erosion

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

9.5.4 Soil Permeability

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

9.5.5 Slope Stability

For the stability of slopes, the 100 year and 50 year return periods earthquake events have been considered, these are equivalent to Magnitude 7 earthquakes with maximum acceleration of 0.20 g and 0.10 g respectively. Instability arising from earthquakes has been analysed by a conventional pseudo-static technique.

The following Factors of Safety have been applied:

static loading conditions including scoured face at the foot of the revetment	1.5
seismic loading and falling the stage (see also Appendix 2.7 of the Design and Construction Management Report).	1.1 generally

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

From the results of numerous physical model tests and reported experience in India and elsewhere, it has been concluded that it is reasonable to assume that the slope of the launched apron is unlikely to be steeper than 1V:2H. With this configuration and the base of the apron set at 10 m below LWL it was found that a revetment slope of 1V:3.5H gave acceptable factors of safety.

9.6 Scour Depth

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

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

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

9.7 Planform for Bank Stabilisation Structures

9.7.1 Introduction

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

9.7.2 Definitions

In this context groynes are defined as hardened structures that protrude into the main channel, when constructed, with the primary objective of deflecting erosive flows away from

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

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

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

9.7.3 Choice Between Groynes and Hard-Points

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

It has been noted in Section 9.7.2 above that unless complementary groynes are provided on the opposite bank (or for some other reason it is erosion resistant), the construction of one or more groynes on one bank of a river will normally result in erosion of the other bank to an equal extent. The concept that groynes result in a net land gain may therefore be illusory. In this respect, the environmental advantage of a groyne over a hard-point, particularly in terms of social benefit, is therefore limited to situations where the treatment of both banks is appropriate.

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9.8 Revetment Structures

9.8.1 Layout in Plan

For the reasons described in the previous section, hard-point revetment structures are the principal form of river bank stabilisation measure proposed for the short term and long term works.

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

The layout of a typical hard-point is shown in Figure 9.1. The length of straight revetment is such as to ensure that there will be an acceptably low risk of the embayment, or any re-entry upstream of the hard-point, threatening the integrity of the cross-bar. The cross-bar is angled back in plan to decrease further the risk of erosion from the upstream side. The cross-bar is lightly armoured against wave action from out-of-bank flow by brick mattressing on its upstream side, for the whole length back to the BRE, and for the 100 m nearest to the river bank on the downstream side.

The revetment is returned by upstream and downstream terminations. As explained earlier, the upstream termination will be subjected to higher flow velocities than the straight revetment and downstream termination, and hence is provided with heavier armouring, as shown in Figure 9.2

The layout at Sirajganj (Figure 8.2) with more than 2 km of standard revetment, is a special case. It has a conventional downstream termination, but the upstream termination, which incorporates the existing Ranigram Groyne, will be armoured as appropriate to the nose of a groyne.

9.8.2 Typical Revetment Section

The main elements of the revetment design are:

- o a formed slope that is designed to be stable under normal combined earthquake loading and drawdown conditions.

- o A geotextile fabric laid on the slope that is designed to permit drainage of the underlying soil while preventing migration of soil particles under differential pressures induced by wave action, turbulent river currents and soil-water flow.

- o An armour layer designed to hold the geotextile firmly in position and therefore capable of resisting the forces induced by high velocity flow and wave action; it must be sufficiently robust and durable to withstand abrasion due to sediment laden water and inter-block movement.

o An apron, commonly called a falling apron or launching apron, consisting of armour material placed at the toe of the slope; this acts as a stockpile that is drawn upon through a natural bed armouring process when unusually deep scour develops off the toe of the revetment.

The basic principles on which the design of the revetment for the Phase 1 Priority Works are based are set out in Appendix 2.1 of the Design ad Construction Management Report, and further details are given in Appendix 2.3 to that report. A typical revetment section in shown in Figure 9.3.

The crest level is determined from the 100 year return period water level, derived from the 1-D hydrodynamic modelling programme described in Chapter 3, with a 1.0 m allowance for freeboard. A single layer of placed armouring extends from the concrete crest wall to the lowest water level likely to be experienced during construction, LWL + 2 m. From that level to the apron setting depth, armouring will be dumped to form the equivalent of two layers.

The apron setting level has been determined from a combination of geotechnical considerations (slope stability) bearing in mind that the scour slope will be steeper than the armoured revetment section, and a reasonable maximum dredging depth. As explained in section 4.7, there is only a relatively low probability of depths exceeding this level. The quantity of armour material in the apron is adequate to protect the scoured slope to design scour depth with a thickness of armouring in excess of two layers.

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

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

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

Cross sections through the crest and cross-bar are shown in Figure 9.4.

9.9 Filter Layer Design

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

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

Geotextiles have been extensively used in other countries for this purpose and under similar environmental conditions; good guidelines for the design and construction of revetments incorporating geotextiles have been published by PIANC (1987). Further advice, particularly regarding material properties, was sought from leading manufacturers and the conclusions were set out in a BRTS Technical Note on Geotextile Selection issued in February 1992, and reproduced as Appendix 2.2 of the Design and Construction Management Report.

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

9.10 Selection of Armour Material

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

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

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

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

The Priority Works are designed with concrete block armouring (brick or stone aggregate) above LWL + 2m, and alternatives of concrete blocks or rock below that level. Hence three alternative tenders are required - concrete blocks with brick aggregate, concrete blocks with stone aggregate, and quarried rock (below LWL + 2 m with the cheaper type of concrete block above).

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

The same procedures may be adopted for the Phase 1B and 1C works, and subsequently for the long term works, unless a clear trend towards a particular alternative emerges in the meantime.

9.11 Construction Methods

9.11.1 River Conditions

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

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

During the six months of lower flows, conditions are most favourable for underwater work for 4 to 5 months, but with good organisation and management and careful planning, productive activity should be possible for the full six months in most years. Even at low flows the sediment levels in the river make visibility under water poor and divers will be obliged to work largely by touch. Navigation at this time of the year is liable to be difficult for larger conventional vessels and considerable dredging may be necessary in order to maintain regular direct water access to the works. Contractors will therefore have to adopt a flexible approach to transport with the siting of wharfs decided only as the river levels begin to fall and the channel patterns and char locations begin to become more firm.

With only limited time available for working in the river, it will be necessary for contractors to build up stocks of materials during the monsoon season adequate to meet the heavy demand once working in the river becomes a practicable proposition.

9.11.2 Slope Preparation

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

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

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

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

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

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

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

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

9.11.3 Placement of Geotextile and Armour Material

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

The order of construction below water level will have to be (1) place the geotextile in the form of geotextile/bamboo fascine mattresses on the lower slope together with the ballast layer, (2) dump at least part of the apron material to anchor the bottom of the geotextile, which extends 2 m under the apron for this purpose, (3) place armour material on the lower slope, (4) complete the placing of the apron. It may be that part of the slope armour placing may be carried out by land based equipment but the apron material will almost certainly have to be dumped from pontoon/barge. Placing of the slope material will require a considerably higher standard of control than for the apron material.

9.11.4 Crest and Access Road

Where fill is required to form the revetment and is placed hydraulically, consolidation will occur swiftly. The material will be predominantly sandy, and the Specification requires that measures be taken to avoid the inclusion of silt. The fill will therefore be relatively free draining and will quickly become trafficable for further construction activities.

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

The design includes a crest retaining wall and access road, and in view of the unsightly appearance which would result were settlement to occur, construction should be left as long

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as practicable after filling. It is planned that settlement be monitored and that the crest works be completed only when it has reduced to an acceptable level.

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

9.12 Construction Quality Control

Under the fourth edition of the Conditions of Contract for Works of Civil Engineering Construction published by the Federation International des Ingenieurs-Conseils (FIDIC-IV) the Contractor is responsible for the quality of his own work and for ensuring that the end product is in accordance with the Specification. The Specification requires that he draws up a Quality Plan in accordance with BS 5750, implicit in which is that he will have to prepare written Procedures which will cover the sequence of operations to be followed for each key activity; and the checks and controls that will have to be built in to ensure that tasks are completed in the correct sequence and that approvals are obtained where required before proceeding to the next task.

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

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