Call- 835 / FAP-21/22

GOVERNMENT OF PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANIZATION

FEDERAL REPUBLIC OF GERMANY

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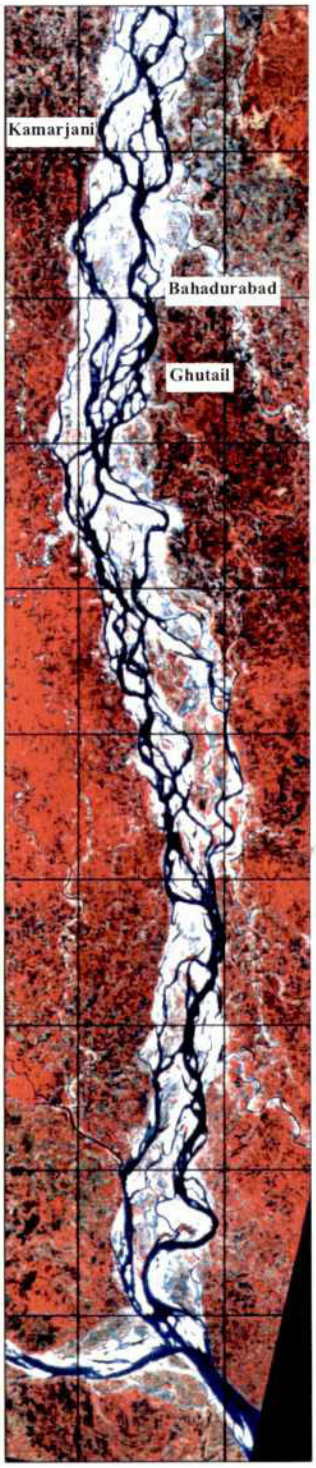

BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT FAP 21/22

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FINAL PROJECT EVALUATION REPORT

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VOLUME I

Main Report
Part A: Bank Protection Pilot Project
Part B: River Training (AFPM) Pilot Project

DECEMBER 2001

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JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE
CONSULTING CONSORTIUM FAP 21/22

RHEIN-RUHR ING.GES.MBH, DORTMUND/GERMANY
COMPAGNIE NATIONALE DU RHONE, LYON/FRANCE
PROF.DR. LACKNER & PARTNERS, BREMEN/GERMANY
DELFT HYDRAULICS, DELFT/NETHERLANDS

In association with:
BANGLADESH ENGINEERING &
TECHNOLOGICAL SERVICES LTD. (BETS)
DESH UPODESH LIMITED (DUL)

BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT FAP 21/22

FINAL PROJECT EVALUATION REPORT

VOLUME I

MAIN REPORT

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PILOT PROJECT FAP 21/22

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MAIN REPORT

DECEMBER 2001

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FINAL PROJECT EVALUATION REPORT

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PREFACE

Bangladesh with an area of 144 000 km^2 and about 130 million people is the most densely settled country on earth. It is characterized by a river system with a total length of about 22000 km (see Fig. next page), among them some of the largest rivers of the world, e.g. the Brahmaputra/ Jamuna, the Ganges and the Meghna. It is periodically inundated by monsoon floods which cover about 20 % of the territory every alternate year and about 37 % of the territory once every tem years. However, the record was broken in 1987 when 40 % of the country was flooded and in 1988 when more than 60 % of the entire territory was inundated.

After these two disastrous floods, Bangladesh received world-wide offers of help to find long-lasting and safe solutions of flood control and drainage improvement. The extent of these offers necessitated co-ordination and were assigned to the World Bank. The role of the World Bank was defined in the communiqué issued by the G 7 Economic Summit in Paris in July 1989; and during a meeting of international donors in London in December 1989 a Flood Action Plan (FAP) was discussed and decided upon. The primary aim of the Flood Action Plan was the development of the country by reducing the consequences of natural disasters. It consisted of 26 planning and support studies in the main regions of Bangladesh, supporting the country's activities to promote improved structure design and implementation as well as non-structural measures. The activities were co-ordinated by the Flood Plan Coordination Organization (FPCO) which passed over the responsibilities to the Water Resources Planning Organization (WARPO) in January 1996. Besides the technical feasibility of the various measures, special attention was paid to the associated socio-economic and ecological effects. Negative interferences between measures and the rivers' ecosystem had to be kept to a minimum.

Within the component FAP 21, the performance and stability of permanent protection structures against bank erosion were investigated, whereas in FAP 22 temporary and recurrent interventions in the active flood plains of the Jamuna river were studied and tested regarding their efficiency and technical feasibility to control bank erosion by comparatively low-cost measures.

Both components, FAP 21 and FAP 22 were jointly financed by Germany and France through the donor agencies Kreditanstalt für Wiederaufbau (KfW) and Caisse Française de Développement (CFD, in 1998 renamed Agence Française de Développement AFD). The respective contracts were awarded to the Consulting Consortium Jamuna Test Works Consultants (JTWC), involving

- Rhein-Ruhr Ingenieur-Gesellschaft mbH (RRI), Germany, as lead partner;
- Compagnie Nationale du Rhône (CNR), France;
- Prof. Dr. Lackner & Partners GmbH (L&P), Germany, and
- Delft Hydraulics (DH), The Netherlands

in association with

- Bangladesh Engineering and Technological Services Ltd. (BETS), Bangladesh, and
- Desh Upodesh Ltd. (DUL), Bangladesh.

As per Terms of Reference, the Consultancy Services for both FAP components were to be performed in two phases

- Planning Study Phase (Phase I), and
- Test and Implementation Phase (Phase II).

Phase I of FAP 21 started in December 1991 and was completed as per contract in January 1993 with submission of the Final Planning Study Report. The Test and Implementation Phase started in June 1993 and was originally scheduled to be completed by end of 1998. However, in 1998 it was decided to build a third test structure using the remaining funds of the project. In order to allow for a test and monitoring period of at least three years, the completion date was finally fixed for the end of December 2001.

Phase I of FAP 22 started together with FAP 21 in December 1991 and was completed as per contract in January 1993 with submission of the Final Planning Study Report. After a review of the proposed measures by a group of international experts in 1993, the implementation of newly developed low cost measures started in 1996 and continued till 1998.

Due to different character of the two FAP components, these are discussed separately in Part A (FAP 21) and Part B (FAP 22) of this Final Project Evaluation Report.

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BANK PROTECTION AND RIVER TRAINING (AFPM)
PILOT PROJECT FAP 21/22

FINAL PROJECT EVALUATION REPORT

MAIN REPORT – PART A

BANK PROTECTION PILOT PROJECT
FAP 21

DECEMBER 2001

FAP 21 – BANK PROTECTION PILOT PROJECT

FINAL PROJECT EVALUATION REPORT

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ATTACHMENTS

LIST OF ACRONYMS

AFD	-	Agence Francaise de Dévelopment
BS	-	British Standards
BWDB	-	Bangladesh Water Development Board
CC	-	Cement Concrete
DHW	-	Design High Water
DIN	-	Deutsche Industrie Norm (i.e. German Industrial Standard)
DM	-	Deutsche Mark
EIA	-	Environmental Impact Assessment
EIRR	-	Economic Internal Rate of Return
FAP	-	Flood Action Plan
FPCO	-	Flood Plan Coordination Organization
FPL	-	Flood Plain Level
GoB	-	Government of Bangladesh
GPS	-	Global Positioning System
HFW	-	High Flood Water
JTWC	-	Jamuna Test Works Consultants
KfW	-	Kreditanstalt für Wiederaufbau
LC	-	Loading Class
MCA	-	Multi Criteria Analysis
O&M	-	Operation and Maintenance
PWD	-	Public Works Department (datum level)
RRI	-	River Research Institute, Faridpur
SHW	-	Standard High Water
SIPPS	-	Strategy for Identification of Priority Protection Sites
SLW	-	Standard Low Water
SMP	-	Strategic Master Plan
SPS	-	Standard Protection Structure
Tk	-	Taka
US$	-	US Dollar
WARPO	-	Water Resources Planning Organization

LIST OF SYMBOLS

d_{sm}	-	Maximum scour depth	(m)
d_{sd}	-	Design scour depth	(m)
D	-	Diameter of sediment particles on river bed	(m)
D_{50}	-	Median diameter of sediment particles on river bed	(m)
D_n	-	Dimensions of cubic concrete block	(m)
H_{des}	-	Design wave height	(m)
h_0	-	Water depth at deepest point of cross-section (thalweg)	(m)
L_g	-	Effective groyne length, i.e. length of groyne projected onto a line perpendicular to the bankline	(m)
T	-	Design wave period	(s)
$\bar{u}_{25}$	-	Cross-sectionally averaged flow velocity for discharge with a return period of 25 years	(m/s)
$\bar{u}_{des}$	-	Design depth averaged flow velocity	(m/s)
W_{fa}	-	Width of falling apron	(m)

Other symbols are explained in the text at their utilisation.

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GLOSSARY

TERM	DEFINITION
bed protection	layered systems placed on filters on a horizontal surface as protection against hydraulic forces and scouring
char	island, sand bank and floodable area adjacent to the banks
cover layer	outer protective layer of an embankment revetment or a bed protection
cross bar	an impermeable structure protruding into the river flow
falling apron	multi-layer system of granular material placed directly on the existent subsoil or riverbed
filter	one-layer or multi layer system of well graded granular material or a geotextile filter mat or a combination of both
gabions	mattresses and rectangular baskets made from galvanised/PVC-coated steel wire mesh and filled with loose material such as boulders, bricks etc.
geotextile filter	synthetic fabric (woven, non-woven, needle punched) applied as a filter or used in tailored geotextile systems (mattresses, etc.)
groyne	a structure, either permeable, impermeable or as a combination of both protruding into the river flow
hydraulic loads	forces due to action of water (hydrostatic or hydrodynamic)
khoa	brick chips (used as concrete aggregates and filter material)
launching apron	integrated and articulating mattress system placed on prepared slopes above and below water or in horizontal excavation well above SLW
mouza	official land owner map
revetment	layered systems placed on filters on a sloping surface as protection against hydraulic forces and scouring
rip-rap	layer of loose stones acting as cover layer in an embankment revetment, a bed protection or a falling apron
scour	removal of soil particles by current or wave induced shear forces
spur	local expression for groyne
toe protection	systems to protect the toe of an embankment against instability due to erosion/scouring

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CHRONOLOGY OF THE FAP 21 PROJECT COMPONENT

YEAR	EVENT
1991	
May 06	Presentation of Proposal
October 14	Signing of Consulting Agreement
December 01	Date of commencement of Consulting Services (Phase I)
1992	
January 13	Start of expatriate staff deployment in Bangladesh
March 14	Technical Report No. 1 (Phase I) on pre-selection of test areas
March 19	Coordination meeting on pre-selection of test areas with FPCO
March 21	Submission of the Inception Report
March 21	Approval of pre-selected test areas
March 28	Signing of subcontract for geotechnical investigations
April 02	Signing of subcontract for topographic surveys
April 08	Signing of subcontract for hydrographic surveys
April 04 to 19	Study Tour to Europe including attendance of 5th Symposium on River Sedimentation, Karlsruhe, Germany
May 22 to June 06	Braided Rivers Study Tour to the Yellow and Yangtze Kiang rivers in China and the Mississippi River in USA
July 16	Submission of the Interim Report
July 26	Signing of subcontract for physical model investigations
November 11	Signing of subcontract for additional topographic and hydrographic surveys
December 03	Signing of subcontract for additional physical model investigations
1993	
January 18	Submission of the Draft Final Planning Study Report
January 06 to 07	Appraisal Mission of KfW and CFD
May 05	Letter to Proceed into Test and Implementation Phase (Phase II)
June 01	Start of Test and Implementation Phase
June 12	Signing of subcontract for construction and installation of the Filter Test Rig
June 21	Meeting of FAP Review Committee on Draft Final Planning Study Report
June 30	Submission of the Final Planning Study Report
July 14	Signing of subcontract for physical model tests
July 18	Submission of Final Invoice Phase I
July 28	Signing of subcontract for topographic and hydrographic survey at Kamarjani Test Site
October 31	Signing of subcontract for geotechnical investigations at Kamarjani Test Site
1994	
February 10	Coordination meeting for Kamarjani Test Site with FPCO and BWDB
February 23	Issue of Tender Documents for Kamarjani Test Site
March 20	Pre-bid meeting for Kamarjani Test Site
April 17	Tender opening for Kamarjani Test Site
June 08 to 20	Technical Assessment of Procurement Arrangements of the Consultant
June 14 to 20	Review Mission of KfW/CFD
August 09	Approval of Final Planning Study Report by the FAP Technical Committee

YEAR	EVENT
1994	
September 04	Order to Commence construction works at Kamarjani Test Site
September 07	Signing of subcontract for construction works at Kamarjani Test Site
September 22	Submission of Tech. Report No.1 (Phase II) on physical model tests
September 22	Submission of Tech. Report No.2 (Phase II) on morphological prediction for test areas
September 26	Coordination meeting for Kamarjani Test Site with FPCO and BWDB
Sept. 28 to Oct. 03	KfW mission for definition of Kamarjani Test Site location and discussions on import of geotextile material
October 01	Start of construction works at Kamarjani Test Site
1995	
February 12 to 17	Review Mission of KfW and CFD
February 26	Submission of Tech. Report No. 3 (Phase II) on filter stability investigation
April 16	Issue of Tender Documents for Bahadurabad Test Site
April 18	Submission of Tech. Report No. 4 (Phase II) on falling apron investigation
May 15	Pre-bid meeting for Bahadurabad Test Site
May 20 to 25	Audit of the Project (Kamarjani Test Site)
May 30	Completion of construction works at Kamarjani Test Site
June 11	Tender opening for Bahadurabad Test Site
August 31	Order to Commence construction works at Bahadurabad Test Site
September 10	Coordination meeting for Bahadurabad Test Site with FPCO
September 20 to 26	KfW mission for definition of Bahadurabad Test Site location
September 30	Signing of subcontract for construction works at Bahadurabad Test Site
December 01	Start of construction works at Bahadurabad Test Site
1996	
February 01	Suspension of construction works at Bahadurabad Test Site
March 12	Submission of Tech. Report No. 5 (Phase II) on additional model tests
March 20	Submission of letters of FORCE MAJEURE to WARPO for both Test Sites
April 22	Proposal for Final Implementation of Revetment Test Structure at Bahadurabad Test Site
May	Adaptation Kamarjani: Start of piling works
June 26 to July 03	Review Mission of KfW and CFD
July 18	Proposal for Modification of Consulting Services
September 30	Submission of Monitoring and Adaptation Report 1995, Kamarjani Test Site
November 13 to 17	Technical Review Mission of KfW and CFD
November 26	Resumption of construction works at Bahadurabad Test Site
December	Adaptation Kamarjani: Completion of piling works
1997	
March 02	Approval of extension of the monitoring period up to December 31, 1999
May 31	Completion of construction works at Bahadurabad Test Site
June 20 to 29	Technical Assessment of Procurement Arrangements of the Consultant
July 11 to 19	Audit of the Project (Kamarjani and Bahadurabad Test Site)
July 14 to 21	Technical Review Mission of KfW and CFD

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YEAR	EVENT
1998	
May 05	Submission of Tech. Report No. 6 (Phase II) on additional model tests
July 14 to 23	Technical Review Mission of KfW and AFD
September 09	Submission of Monitoring and Adaptation Report 1996, Kamarjani Test Site
December 23	Proposal for modification of Consulting Services for Ghutail Test Site
1999	
March 01 to 07	Technical Review Mission of KfW and AFD
March 23	Submission of Monitoring and Adaptation Report 1997, Kamarjani Test Site
March 23	Submission of Monitoring and Adaptation Report1997, Bahadurabad Test Site
	Revised Proposal for Modification of Consulting Services for Ghutail Test Site
June 23	Signing of subcontract for construction works at Ghutail Test Site
December 17	Start of construction works at Ghutail Test Site
December 23	Approval of extension of the construction and monitoring period up to December 31, 2000
2000	
February 05	Submission of Monitoring and Adaptation Report 1998, Kamarjani Test Site
February 05	Submission of Monitoring and Adaptation Report1998, Bahadurabad Test Site
Feb. 26 to Mar. 06	Technical Review mission of KfW
April 15	Handing over of the Groyne Test Structure at Kamarjani to BWDB
May 25	Completion of construction works at Ghutail Test Site
July 01	Inauguration of Ghutail Test Structure by GoB
Aug. 29 to Sept. 19	Technical Assessment of Procurement Arrangements of the Consultant for Ghutail Test Site
September 12 to 19	Audit of the Project (Ghutail Test Site)
December 14	Handing over of the Revetment Test Structures at Bahadurabad and Ghutail to GoB
2001	
April 13 to 20	Technical Review mission of KfW and AFD
December 08 to 15	FAP 21 - Workshop in Dhaka
December	End of the Project

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1 PROJECT FRAMEWORK OF FAP 21

1.1 OVERALL OBJECTIVES AND METHODOLOGY

1.1.1 Objectives

The objectives of the FAP 21 Project were to find improved solutions for protection measures against bank erosion by designing and investigating different structure types at moderate safety levels. It was a condition to utilize local materials and construction methods wherever possible. After implementation of the proposed test structures at selected locations along the banks of the Jamuna river, the structural properties and the development of the neighbouring river morphology had to be monitored for a period of at least three years. Based on the experiences gained during the structure implementation and the monitoring period, optimised design criteria as well as cost effective construction and maintenance methods were to be developed ultimately leading to future standards most appropriate for the prevailing conditions at the Jamuna and other major rivers of Bangladesh.

1.1.2 Methodology

The general steps of the methodology during the Planning Study Phase (Phase I) and the Test and Implementation Phase (Phase II) are briefly described as below (also refer to Fig. 1.1-1).

To allow for a better understanding of the morphological conditions and the typical properties of the Brahmaputra/Jamuna river, earlier reports and investigations were scrutinised and experiences gained from other large alluvial rivers in the world with braiding characteristics were studied. In addition, a study tour of experts involved was undertaken including visits to the Mississippi river in the USA and the Yellow river and the Yangtze Kiang in China.

The selection of sites for the implementation of the test structures was one of the focal points of the Project. Since the Jamuna River alters its course often, a continuous verification of the predicted possible future developments in the test site areas was required.

Topographic surveys as well as hydraulic measurements and subsoil investigations were carried out to provide the data bases for setting up and calibrating physical and mathematical models of the test structures. These were used to determine the relevant design parameters. Based on the results of the model tests, two different structure types employing various structural components and materials were selected for implementation.

After completion of the structures, a comprehensive monitoring programme was carried out. Following the analysis of the river response and the structural performance incl. the experience gained during the construction works, conclusions and recommendations for future erosion protection measures in Bangladesh were drawn.

PLANNING STUDY PHASE

INPUT
Screening of data and experience

Available morphological, hydrological and meteorological data
Experience from other rivers with similar properties
Traditional/ established construction techniques in Bangladesh
Available materials and equipment

PROCESSING
Site specific investigations

Hydrological and morphological surveys
Subsoil properties
Physical / numerical modelling
Expert judgement

OUTPUT
Conclusions for the Test and Implementation Phase

Decision on structure types and materials
Selection of test locations
Hydraulic and morphologic boundary conditions

TEST AND IMPLEMENTATION PHASE

DESIGN

Design philosophy for test structures
Projected safety levels and design methods
Decision on construction techniques
General layout and detailed design of test structures

IMPLEMENTATION

Preparation of working drawings and tender documents
Contract award
Management and supervision of construction works
Controlling (budgetary means)

EVALUATION

Monitoring of river bathymetries and hydraulic boundary conditions
Observation of scour development
Inventory of structural damages
Analysis of failure mechanisms
Comparison of measured/calculated hydraulic loads

OUTPUT

Safe and economic design methods and procedures
Standard structure types
Suitable construction methods/materials
Strategy for project implementation

Fig. 1.1-1: Project framework FAP 21

1.1.3 Design Philosophy

Existing design methods are usually based on empirical approaches. To allow for an adoption of these formulae to the very specific hydraulic and morphologic environment at the Jamuna River, the critical hydraulic conditions had to be investigated. Hence, the test structures were designed at a safety level at which certain partial damages of the structures were expected without affecting the overall stability. In a first step, boundary conditions and design criteria had to be determined accordingly. To develop safe and economic design standards for future erosion protection measures, it was decided to choose the lowest tolerable dimensions of individual structure components and to investigate their sensitivity against hydraulic loads (currents and waves).

1.2 THE STUDY PHASE

1.2.1 Objectives

The objectives of the Study Phase were to investigate and analyse the morphological behaviour and development of the Jamuna river and to select suitable locations for the planned test structures. Various appropriate measures against bank erosion had been investigated and the suitability of local materials and construction methods had been analysed. Alternative design solutions were studied and the design criteria and safety levels were determined according to the design philosophy of the Project. Finally, the preliminary design of the first two selected test structures was prepared.

1.2.2 Studies and Investigations

The following technical investigations and studies have been performed:

- **Morphological studies** were necessary to understand the morphological behaviour of the Jamuna river and to predict possible future developments of its planform with regard to the determination of potential test site areas on both banks of the river. The morphological changes near these pre-selected test site areas were observed and finally two areas were selected which were found suitable for the construction of the test structures with a high probability of being attacked during the following years.

- **Geotechnical investigations** comprised field and laboratory tests to obtain general information on the prevailing soil stratification and properties, including ground water levels within the test site areas. This was required for stability calculations of the test structures.

- **Topographic and hydrographic surveys** had to be performed because already existing topographical maps and surveys did not provide sufficiently detailed and up-to-date information on the particular pre-selected test areas. Hence, extensive surveys were required to obtain appropriate maps in order to study the extent of bank erosion in those areas which was essential for the preparation of physical and mathematical models as well as for the evaluation of alternative structural design and alignments of the test structures.

- **Hydrometric measurements** to provide data on flow velocities and flow directions at selected locations in the test site areas as a basis for the preliminary structure design.

- **Mathematical modelling** was applied to determine the flow field in the vicinity of the planned test structures based on the surveyed bed geometry and predicted future bank lines. The results were used to define the maximum approach flow velocities which were required for the design of the test structures.

- **Physical model tests** were performed at the River Research Institute in Faridpur to determine the maximum flow velocities and the expected scouring in the vicinity of the structures. The layout of the test models was defined on the basis of field surveys and predictions regarding the future morphological development and bank erosion in that particular area.

In addition to the technical investigations, **socio-economic**, **agro-economic** and **anthropological surveys and investigations** were carried out for both test sites because the ultimate objective of all bank protection works is to contribute to the benefit of the people concerned, either directly by protecting their lives and properties, or indirectly, by protecting infrastructure and other assets. The socio-economic activities included discussions and active participation of the local population on land acquisition problems and resettlement of households. In order to take environmentally sound decisions, an environmental assessment was carried out for both test site areas, to identify and possibly prevent from negative impacts.

1.2.3 Proposed Structural Solutions

Based on the results of the physical and mathematical model tests and other technical and non-technical investigations, it was decided to build a series of 6 Groynes at Kamarjani Test Site on the right bank and a Revetment Structure downstream from Bahadurabad ghat on the left bank of the river.

Later, in 1999, it was decided to use the remaining funds for the construction of a second Revetment Test Structure at Ghutail Bazar about 3.5 km downstream from Bahadurabad.

1.3 THE IMPLEMENTATION PHASE

1.3.1 Preliminary Remarks

The Consultant's services during the Test and Implementation Phase (Phase II) comprised all engineering and management tasks relating to the planning and design of the test works. Furthermore, it was part of the contract to carry out monitoring and in case of damages, any required adaptation/repair works in the subsequent years during the project period. At the end of the contract period the Pilot Structures were to be handed over to BWDB.

As the implementation of the structures were postponed by one year, the results of the planning study had to be updated. Due to the rapid changes of the bankline, the pre-selected test sites had to be confirmed. Additional site investigations and physical model tests were also carried out to verify the final location and the design of the test structures. Nevertheless, due to delays during the land acquisition process the uncompleted revetment structure was partly destroyed in the dry season 1995/96 and could not be completed before monsoon 1997.

The activities of Phase II can be summarized as follows:

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a) Preparation and Implementation
- additional studies and field investigations;
- final determination of the test sites;
- tender documents including preparation of final design of the test structures, tender drawings, working drawings, work schedule, specifications and contract conditions;
- prequalification of contractors;
- tendering and awarding of contracts;
- procurement of key construction material and equipment;
- management and supervision of construction works.

b) Monitoring, Maintenance and Repair
- monitoring of the test structures after completion;
- evaluation of performance of the test structures;
- adaptation of design and construction methods, and
- adaptation and/or repair of test structures if required.

c) Handing over of the test structures to BWDB at the end of the Project.

1.3.2 Implementation

During the appraisal mission of the donors in January 1993, it was decided to start with the design and construction of the groyne field. The implementation services started in June 1993 after approval of the findings of the Planning Study by the Client. The chronological sequence of the implementation was as follows:

- Construction of the Groyne Test Structure at Kamarjani during the dry season 1994/95;
- construction of the Revetment Test Structure at Bahadurabad during the dry seasons 1995/96 and 1996/97, and
- construction of the Revetment Test Structure at Ghutail Bazar during the dry season 1999/2000.

Due to the fact that the implementation of the Revetment Test Structure at Bahadurabad was delayed by one year and the construction of a third test structure at Ghutail Bazar was included, the Project was extended until December 2001.

1.3.3 Monitoring

In order to study the structures behaviour as well as their interaction with the river's hydraulic and morphology, regular monitoring of these structures after their completion was an essential part of the Project.

Monitoring of the Test Structures was aimed to
- detect damages at an early stage;
- understand failure mechanisms, and
- plan suitable adaptation and/or repair works.

In addition to this, bathymetric surveys in the near field and in the neighbouring areas (up and downstream) were used as a basis to investigate the rivers response. The meteorological and hydraulic boundary conditions. Detailed records were taken of the

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- meteorological conditions at the test sites (precipitation, wind speed, wind direction);
- hydraulic loads acting on the structures (water level rise and fall, flow velocities, wave heights);
- bathymetry of the river in the vicinity of the test structures;
- movements of structure components in vertical and horizontal direction;
- deterioration of materials used, and
- damages of structure components.

All observations and activities were documented in a logbook which were used as database for evaluation of necessary repair measures. The logbook and associated records represent a continuous record of events, development of failure mechanisms and the related hydraulic boundary conditions.

The different tasks of the monitoring programme are listed in Table 1.3-1. Apart from daily routine observations, systematic periodical inspection programmes were carried out. However, time intervals between these comprehensive inspections had to be shortened during and after extreme hydraulic conditions.

Type of survey	Parameters
Meteorological Measurements	Wind speed and direction Precipitation Temperature Relative humidity
Hydrologic Measurements	Flow velocity and direction Water level recordings Wave recordings/visual wave observations
Bathymetric surveys	Side scan sonar measurements Sub-bottom profiling Scour development
Land Surveys	Topographic surveys Bankline surveys Char surveys Establishment of benchmarks
Structure inventory	Alignment, pile position Pile recordings Detailed damage surveys during / after the flood period Quantity of floating debris at groynes
Processing and Reporting	Quality control of data Processing and visualisation of critical profiles Preliminary charting Logbook of activities and daily observations

Table 1.3-1: Main components of the monitoring programme

2 SELECTION OF TEST SITES

2.1 PRE-SELECTION OF TEST SITES

The identification of suitable sites for implementation of the test structures along the banks of the Jamuna River was an elemental task of the Project. Therefore, a pre-selection of test areas was made at an early stage of the Project. According to the ToR at least one protective structure should be built on either side of the river. The potential test sites were pre-selected by taking into account the following criteria:

- "certainty-of-attack"
- "something-to-defend"
- accessibility
- availability of data

A primary requirement for the testing of the structures was the "attack" by hydraulic loads for a sufficiently long period of time which had been assumed to be at least three years. The 'certainty-of-attack' criterion (i.e., the relatively high probability of a particular area being subject to more than moderate bank erosion during the Test and Implementation Phase) was analysed by comparison of successive satellite images, involving a considerable part of expert judgement. Due to the difficulties related to the prediction of future bankline development, most emphasis was put on this aspect. It was concluded that the predictions of future morphological developments were to be considered as tentative only and were to be checked and updated regularly in the further course of the Project. The banks of the river erode easily and sometimes result in substantial changes in the river's course during individual monsoon floods. Whenever the location of the bankline attack shift rapidly the locations of the test structures had to be adapted accordingly.

The 'something-to-defend' criterion took into account the population density as well as the use of land and other assets such as private and public infrastructure. In addition, in some cases other devastating large scale changes of the Jamuna River were aimed to be prevented (e.g., changing of the main river bed).

The last two criteria listed above were motivated by more or less practical reasons. A reasonable accessibility of the test site was required to limit the costs related to site facilities and to the transport of materials and construction equipment. The availability of historic information on hydraulic conditions and morphologic developments for specific areas was essential to support recent data sets, employed for the 'certainty-of-attack' criterion.

A list of twelve potential test site areas was prepared (see Fig. 2.1-1) and from these the following shortlist was agreed on the basis of recent bank erosion rates as calculated from satellite images:

- Kamarjani
- Bahadurabad
- Chandanbaisa
- Nakalia

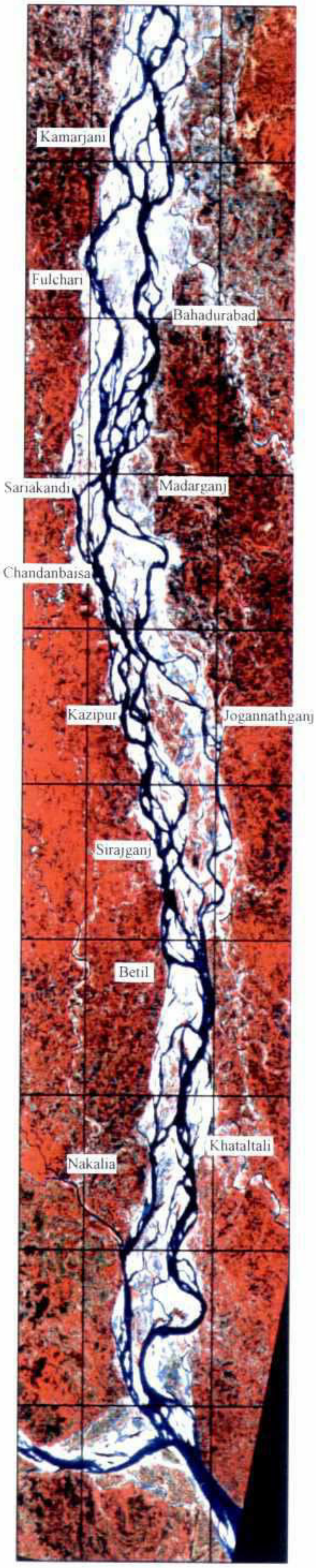
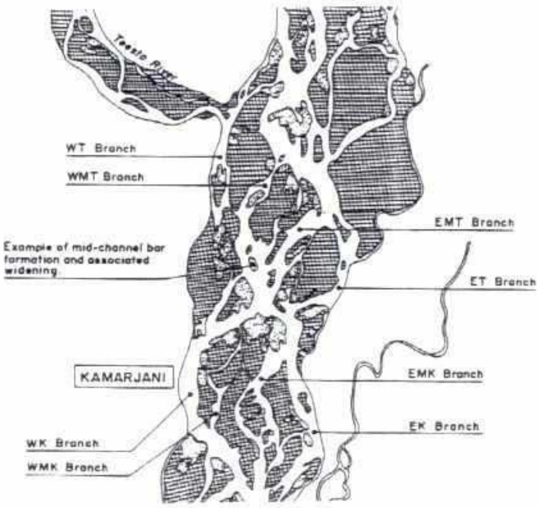


Fig. 2.1-1: Potential test site areas

2.2 FINAL SELECTION

2.2.1 Kamarjani Test Site

The Kamarjani test site area is located at the right bank of the Jamuna River, approx. **25** km downstream from the confluence of the Teesta river (Fig. 2.2-1). In 1993 several major branches were identified in the region where Kamarjani is situated at Western Kamarjani Branch (WK Branch).



- WK Branch (West Kamarjani Branch);
- WMK Branch (Western Middle Kamarjani Branch);
- EMK Branch (Eastern Middle Kamarjani Branch)
- EK Branch (East Kamarjani Branch)
- WT Branch (West Teesta Confluence Branch)
- WMT Branch (Western Middle Teesta Confluence Branch);
- EMT Branch (Eastern Middle Teesta Confluence Branch),
- ET Branch (East Teesta Confluence Branch)

Fig. 2.2-1: Kamarjani Test Site Area (September 1993)

After it had been decided in the course of the appraisal mission in January/February 1993 to postpone the implementation of the first test site at Kamarjani by one year, further morphological investigations and a permanent check of the suitability of the pre-selected test site areas was inevitable. Initially, the test structure was planned to protect the Manos Regulator but substantial erosion made this regulator collapse into the river in July 1993. The bankline retreated by about 1000 meters within three months.

The changes of the river course were analysed by comparison of successive satellite images over a specific period of time. This procedure is amply demonstrated in Fig. 2.2-2. The two images on the left hand side show the satellite images of 1994 and 1995 respectively; whereas the figure on the right hand side shows the processed data, identifying the areas of erosion and accretion. The assessment after the monsoon season in November 1993 confirmed, that Kamarjani would be a suitable area for the construction of test structures in the dry season of 1994/95 because the probability of bank erosion was predicted to be more than 80 % until 1996.

After the loss of the old Manos Regulator, the highest "certainty-of-attack" in the area of Kamarjani shifted to the downstream part of the river bend near Rasulpur. The final test site was selected further upstream near the village Dhutichara, closely north of the new mouth of the rivers Manos and Ghagot. This decision was made to protect the new Manos Regulator and to prevent from a breach of the Jamuna into the Manos and Ghagot rivers. The exact location and the general layout of the implementation proposal together with the results of a bathymetry survey by FAP 24 of March 1995 are shown in Fig. 2.2-3.

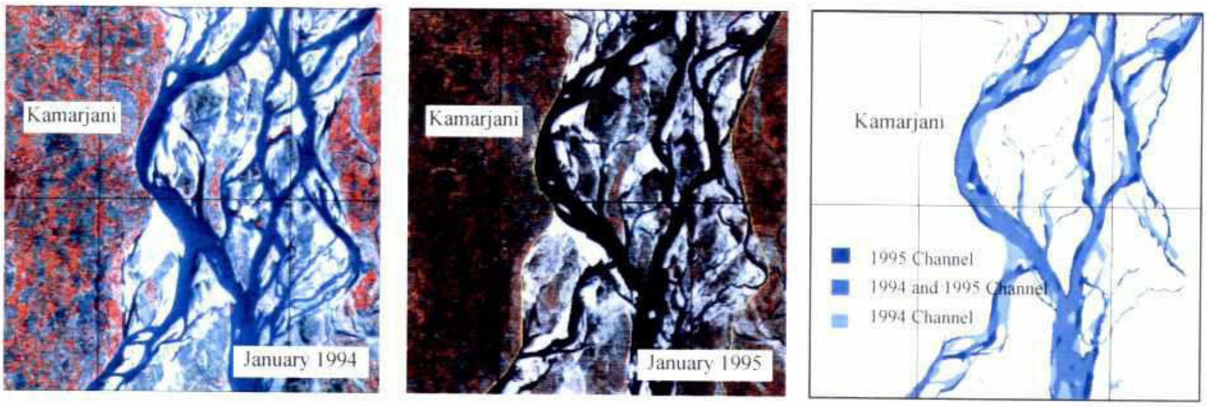


Fig. 2.2-2: Planform changes at Kamarjani between January 1994 and January 1995

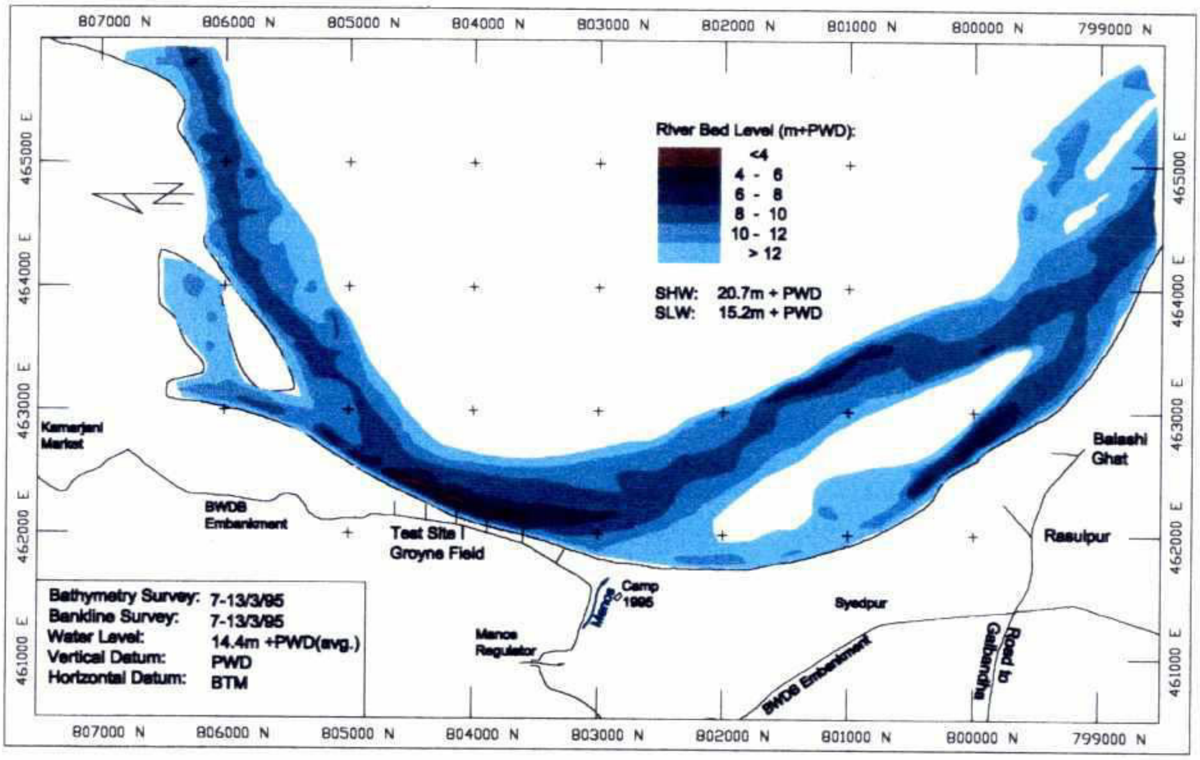


Fig. 2.2-3: Final location of the groyne field at Kamarjani

2.2.2 Bahadurabad Test Site

During the Planning Study, Belgacha (Bahadurabad area) was chosen as the site for the second Test Structure. Since 1990, a major channel had been impinging on the bank at Belgacha about 3 km south of the Railway Ghat, whereas the embayment immediately north of the Railway Ghat had been silting up. Rather unexpectedly new channels were formed in the silted embayment during the flood of 1992, as was observed on a satellite image of the subsequent dry season in 1993. These channels were predicted to grow further and, indeed, one of them developed into a major bend cut-off channel during the flood of 1993. The resulting new major approach channel was more or less straight over a distance of 12 km, thus making various different morphological developments possible. This reduced the predictability of bankline attack because in the absence of a clear bend capricious planform changes easily occur. It was therefore not possible to find any site in the area with a high certainty of attack during the first few years after completion of the structure.

Alternative areas were studied but they often had disadvantages such as interference with ferry operations or the construction of FAP 1 hard points. Bank erosion during the flood of 1994 occurred at the Railway Ghat and about 8 km downstream but not at Belgacha. Despite the low certainty, continued erosion of the Railway Ghat and Kulkandi remained likely in the immediate future and therefore Kulkandi was selected as a test site. The construction of a Revetment Test Structure at this site was started in the dry season 1995/96. Fig. 2.2-4 shows the morphological situation at the beginning of the construction activities in November 1995. In January 1996 the construction had to be suspended due to delays in land acquisition and construction and was postponed until the construction window 1996/97. As a result, the incomplete test structure was exposed to the river's attack during the monsoon season 1996. This was followed by the erosion of the unprotected earth substructure by the river flow. Hence, the location had to be adapted accordingly taking into account the morphological situation in autumn 1996.

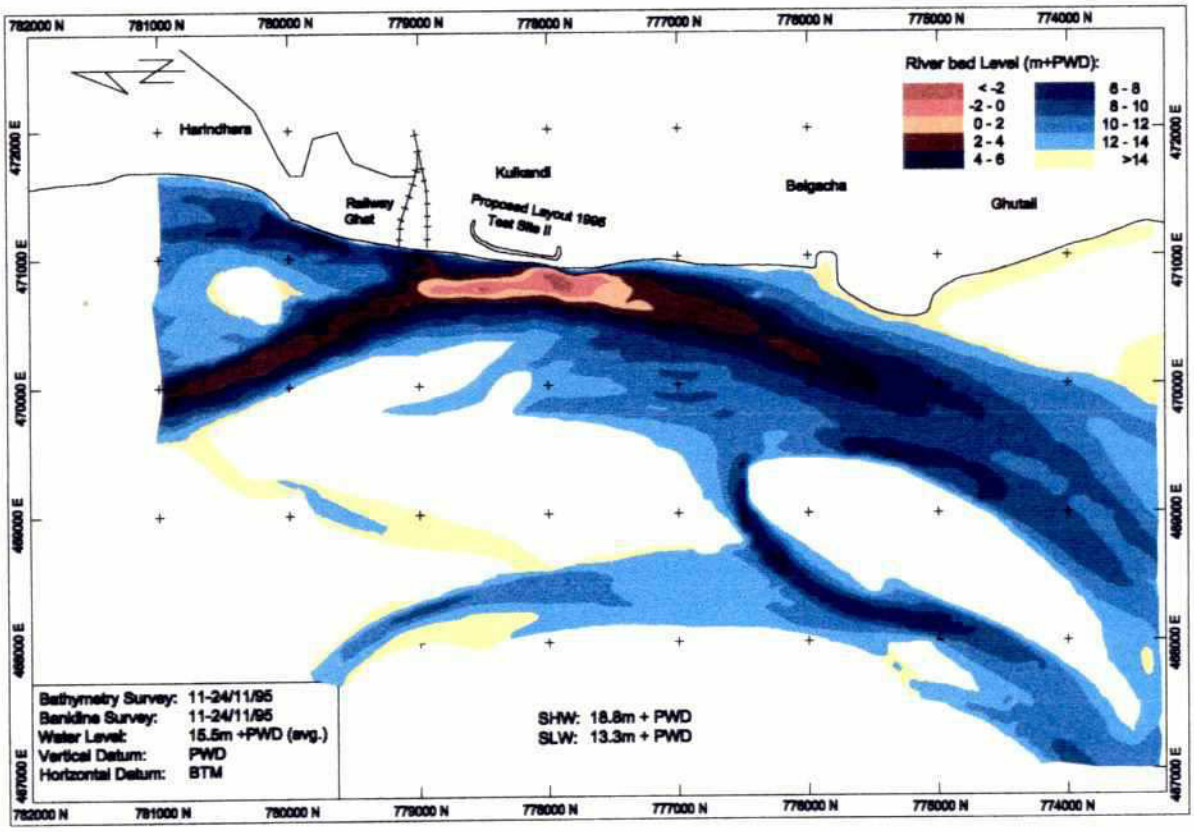


Fig. 2.2-4: Morphological situation at Bahadurabad in November 1995

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2.2.3 Ghutail Test Site

After successful completion of two bank protection structures in the northern reach of the Brahmaputra/Jamuna River, a decision was taken to build a third structure at Ghutail Bazar using the remaining funds of the Project. The decision concerning the third test site was based on the continuing bank line recession in that area and the protection of a prosperous trade market. Furthermore, due to the restricted amount of financial resources an area was selected where the scour depth in front of the structure was assumed to be considerably smaller as compared to the other two test sites. The final position and the structure layout was completed after the preceding flood in October 1999 as soon as the bankline became stable.

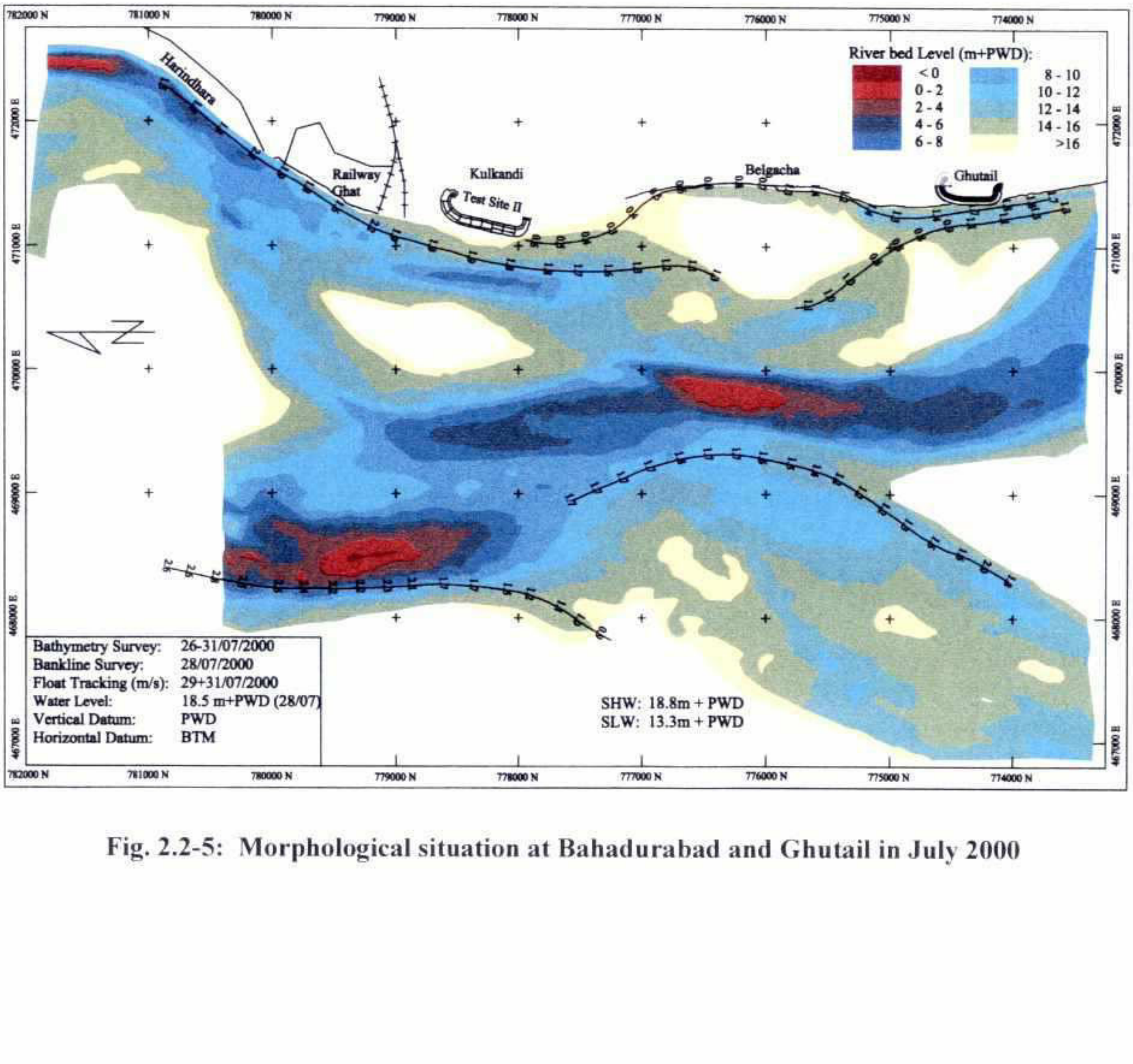

Fig. 2.2-5: Morphological situation at Bahadurabad and Ghutail in July 2000

3 THE GROYNE TEST STRUCTURE

3.1 TEST SITE LOCATION

The Kamarjani area is located on the right bank of the Jamuna River at an approximate latitude of 25°20'N. The area lies at the outer bank of a large migrating bend (see Fig. 3.1-1) and was identified in 1992/93 as a location with high "certainty-of-attack" during the coming years (see Subsection 2.2.1).

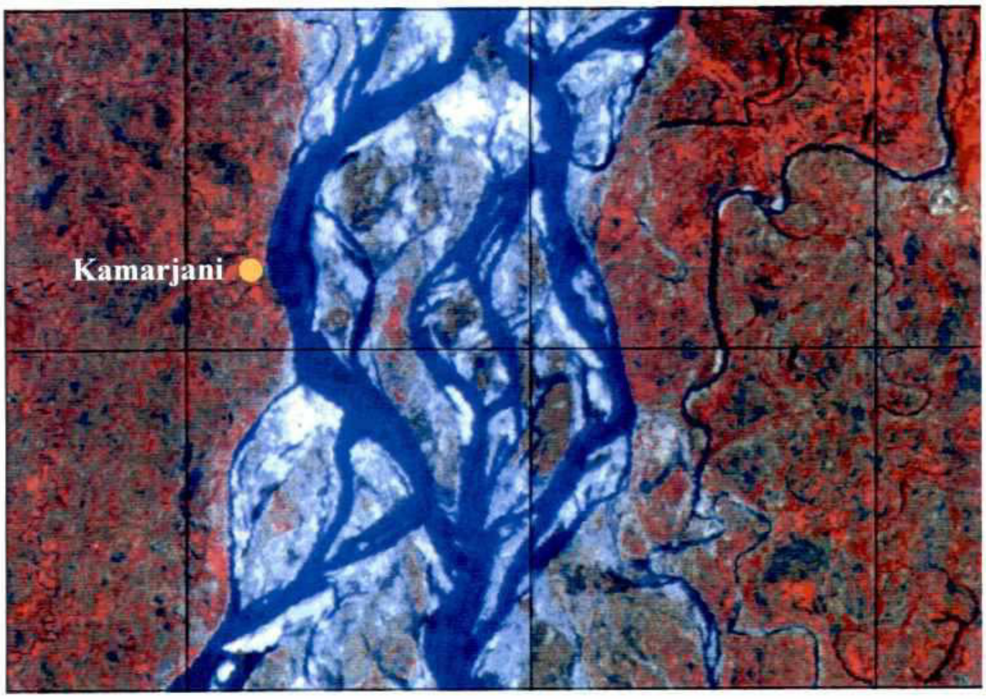

Fig. 3.1-1: Location of the Kamarjani Groyne Field at the right bank of the Jamuna river

3.2 DESIGN CRITERIA AND BOUNDARY CONDITIONS

3.2.1 General Remarks

The design of the groynes followed the concept as per ToR to build cost-effective structures as far as possible using local materials and construction capabilities available in Bangladesh. The implemented test structure comprised 6 combined groynes (connected to the main embankment), consisting of permeable and impermeable sections. The construction works started in October 1994 and were completed by the end of March 1995.

According to the test character, the groynes were purposely designed with a minimum safety level, thus allowing for limited damages during monsoon floods.

The main design parameters are compiled in Table 3.2-1.

Design Parameter			Design Value
High water level		DHW_{25}	22.90 m+PWD (7.70 m+SLW)
Depth and cross-sectional averaged flow velocity	for piles	$\overline{u}_{25}$	3.2 m/s
	for bed protections	$\overline{u}_{2}$	2.3 m/s
Return currents between the groynes			0.5 m/s
Maximum scour depth	downstream from piles	d_{sm}	7.0 m
Design scour depth for piles	with bed protection	d_{sd}	0.0 m
	without bed protection	d_{sd}	4.0 m
Wave height		H_{des}	1.0 m
Thickness of floating debris			1.0 m

Table 3.2-1: Design parameters for groyne field at Kamarjani

3.2.2 Water Levels and Flow Velocities

The Design High Water Level (DHW) was chosen corresponding to the probability of re-occurrence within a 25-year period. Accordingly, the piles of the permeable groynes were designed in consideration of a calculated flow velocity for water levels with a 25-year return period ($\overline{u}_{25}$), to ensure the overall stability even under more severe circumstances. Contrary to this, protective layers of the embankment, bed protections and falling aprons were dimensioned under assumption of a flow velocity calculated for water levels with a 2-year return period ($\overline{u}_{2}$), to avoid over-designing and thus to allow for certain partial damages of structural components, in accordance with the project design philosophy (Section 1.2).

3.2.3 Scour Depths

The most important design parameter for the stability of the groyne piles is the expected scour depth, because this parameter is decisive for their embedment length. The results of the physical model tests suggested that under flow conditions the scour depth at the head of a permeable groyne would be reduced to 20 – 30 %, as compared to an impermeable groyne.

It was also found, that for a permeable groyne the deepest scour (d_{sm}) would not occur directly at the head (as for impermeable groynes) but downstream at a considerable distance from the structure. Consequently, the deepest scour does not have to be considered as design scour depth for dimensioning the embedment length of the piles. Besides this, the depth and location of the scour is obviously influenced by the type of scour protection provided around the structure.

3.2.4 Waves

In absence of recorded data, theoretical calculations were carried out in terms of wind induced waves. A design wave height of 1.0 m was chosen for the Groyne Test Structure. For more details refer to Subsection 4.2.3.

3.2.5 Floating Debris

According to earlier observations, the maximum layer thickness of floating debris was estimated to 1.0 m.

3.2.6 Soil Characteristics and Seismic Loads

During the Study Phase of the Project detailed subsoil investigations were carried out at several potential test site locations along both sides of the Jamuna River. The results had been compared with the data elaborated within other FAP-projects as well as with generally available geological data. It was concluded that subsoil conditions were considerably constant along the Jamuna river. The following main layers were identified:

i) Close to the flood plain surface up to a depth of about 5 m on average: slightly cohesive soil stratum;
ii) below the upper layer until a depth of about 20 m: loosely to medium dense deposited micaceous silty fine sand; and
iii) below 20 m up to a depth of interest for bank erosion prevention measures: medium dense to dense silty fine sand.

Due to the test character of the structures, earthquake loads were not considered.

3.3 LAYOUT DESIGN OF THE GROYNE FIELD

3.3.1 Physical Model Tests prior to Implementation

Physical model tests were carried out at the River Research Institute (RRI) in Faridpur/Bangladesh and at a side arm of the river Rhône at Chanaz/France in order to optimize the configuration and alignment of the groyne field as well as to study the behaviour of falling aprons (made of concrete blocks) as scour protection.

3.3.2 General Layout of the Groyne Field

The groyne field implemented at Kamarjani was initially planned with three main groynes (identified as groynes G-1, G-2 and G-3), two supplementary groynes at the upstream side (groynes G-B/1 and G-B/2) and a further groyne at the downstream side (groyne G-A). The layout of the groyne field is depicted in Fig 3.3-1. To serve as an example, a cross-section of groyne G2 is shown in Fig. 3.3-2.

The total length of the individual groynes varied between 110 m and 130 m. In accordance with the results of the physical model tests, the groynes were designed with a bank-sided impermeable part of about half of the total groyne length and a permeable part extending into the river. In general, the groynes rose above design high water levels. Only groyne G-3 was designed with a lower crest so that it was partially submerged during high water discharges.

The spacing between the individual groynes was chosen at 250 m to 300 m (about 2 to 2.5 times the groyne length), assuming that the bank erosion between the individual groynes could be controlled within tenable limits. The groyne axes point in upstream direction at an angle of 15° (Fig. 3.3-1) to deflect the river's flow away from the bank.

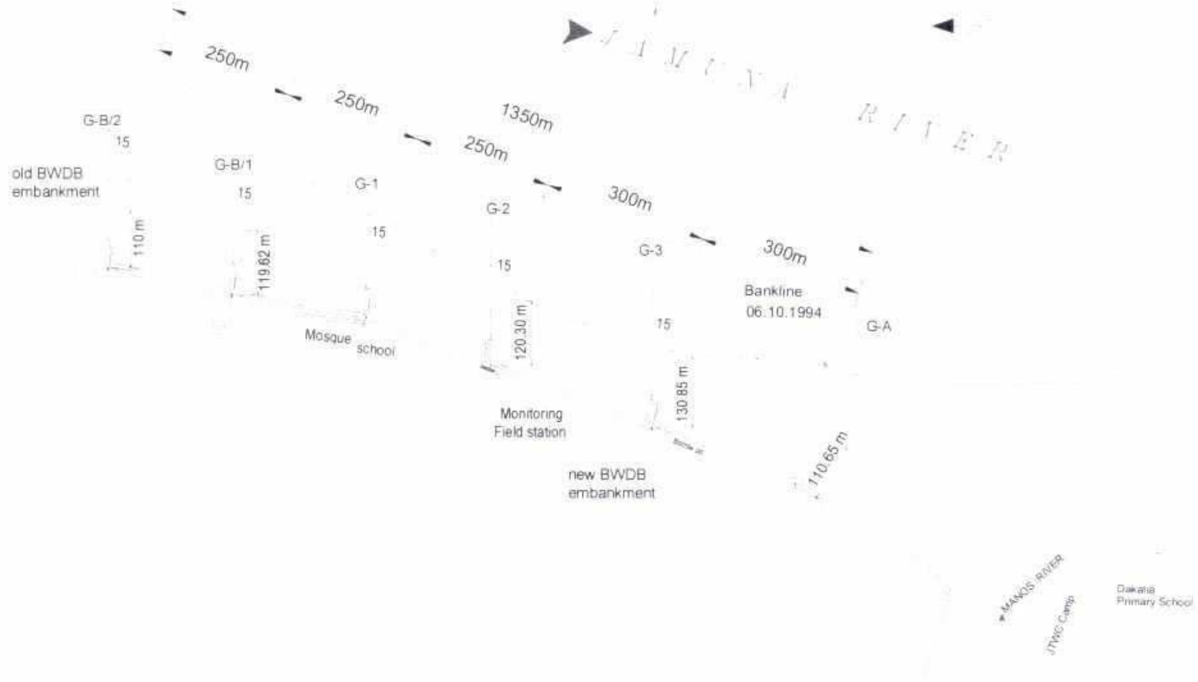

Fig. 3.3-1: General layout of original groyne field

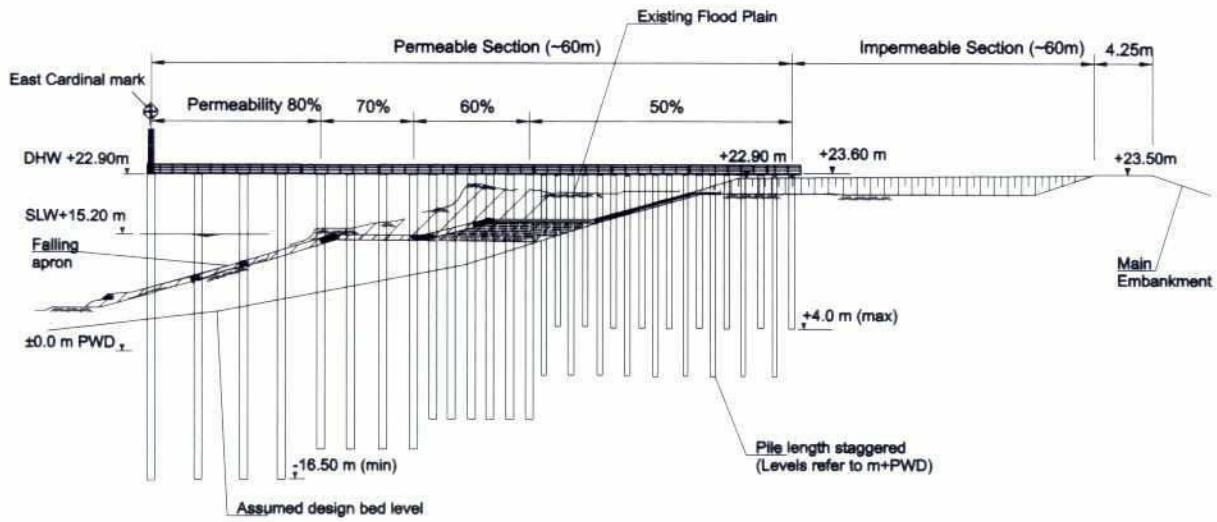

Fig. 3.3-2: Cross section of a semi-permeable groyne (groyne G2, original design)

3.3.3 Main Embankment

The typical layout of groyne fields emphasizes the required link between individual groyne structures and existing main flood embankment to prevent erosion starting at the unprotected groyne root. Due to the fact that at the time of construction no main embankment existed (due to earlier destruction), a new embankment was built under the authority of BWDB.

The river facing slopes of the main embankment were protected by different armour units, such as

- brick mattresses (as per local standard);
- concrete blocks (placed in interlocking pattern);
- rip-rap, and
- Durba grass sods with a footing of Vetiver plantation (only in areas where minor flow velocities were expected).

The protective layers were placed on synthetic geotextile filters and granular filters of different composition respectively. The grass sods were laid on sandy loam, reinforced by geo-jute sheets and nursed initially to encourage growth.

According to the BWDB standard, the landward slopes of the main embankment remained unprotected.

3.3.4 Impermeable Part of the Groynes

The following structural options for the impermeable sections of the groynes were tested:

- Cofferdam of steel sheet piles (groyne G-1);
- cofferdam of concrete sheet piles (groyne G-A), and
- earthdam with slope and toe protection (all other groynes).

The root of the impermeable section was connected to the main embankment, whereas the head of the impermeable part was sloped towards the river to provide a gradual transition between impermeable and permeable groyne sections.

The earthdams were sloped 1 V : 3 H (towards up- and downstream) and were protected by different cover materials; namely rip rap, concrete blocks and brick mattresses, placed on granular or geotextile filters.

3.3.5 Permeable Part of the Groynes

According to the results of the physical model tests, the permeable parts of the groynes were designed to cater for with an increasing permeability of 50 % (at the groyne root) to 80 % (at the groyne head). Due to savings in cost and construction time but very similar functional performance mono-pile structures were given preference over twin- or triple-pile row groynes. Only the groyne heads were designed with twin piles, to cover for accidental impact loads (i.e. ship impact). The following types of piles were selected:

- Steel piles (∅ 711 mm, L=13 / 19.5 / 26 m; ∅ 1016 mm, L=32 m; ∅ 1220 mm, L=36 / 40 m);
- in-situ cast bored concrete piles (∅ 914 mm, L=24 m), and
- pre-stressed spun concrete piles (∅ 500 mm, L=10 / 20 m).

The type and size of piles were chosen with respect to material availability, structural requirements and executional considerations, i.e. expected driving difficulties and construction on-shore or afloat.

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In groyne sections with a permeability of 50 %, the piles were staggered in depth due to the small clearance between the individual piles to prevent problems during pile installation.

The groyne heads were equipped with navigational marks to provide safe passage of inland vessels, crafts, etc.

3.3.6 Bed Protections and Falling Aprons

Within the Pilot Project a comparison between the stability of bed protections and falling aprons in relation to unprotected river beds around the groyne pile-structures was carried out. In this context, bed protections were defined as a suitably dimensioned granular material, e.g. boulders or concrete blocks, placed on a filter), whereas falling aprons were defined as protective materials placed directly on the existent subsoil and riverbed respectively. Falling aprons were also used as toe protection for the revetments.

3.4 MATERIAL AND EQUIPMENT

3.4.1 General Remarks

Bank protection works can generally be executed and completed only within a construction window defined by the low water period between two monsoon seasons. Consequently, advance procurement of materials and equipment has to be arranged to avoid delays in the project implementation. In case of procurements outside Bangladesh, sufficient time for shipping, custom clearance and payment of duties have to be taken in to consideration.

3.4.2 Availability of Construction Materials

Prior to the design phase, the availability of various construction materials in Bangladesh was investigated. It was found that materials of mineral source, like bricks, concrete aggregates, gravel and sand are available or could be produced in the country in suitable quality and sufficient quantities. Cement as well as large boulders and rock for slope protection works have to be imported from neighbouring countries.

Reinforcement steel in standard quality is available in the market up to ∅ 28 mm. Wire mesh is produced locally, hand made from galvanized or non-galvanized steel wire (Photo 3.4-1). Availability of new structural steel is limited to flat steel, small L-profiles and U-profiles. Other and heavier profiles were found, as used material, in limited quantities.

Tubular steel pipes are available in the water and gas sector in standard qualities and dimensions up to ∅ 200 mm. Larger pipes can be rolled and welded in the country but with restrictions in dimensions. However, due to time constraints the steel piles for the test structure were imported from France.

Up to a limited water depth and hydraulic loads, locally produced concrete piles are an attractive alternative to steel piles. However, a general disadvantage of concrete piles is the lower bending moment capacity and the fact that their embedment length cannot be increased, once constructed in comparison with steel piles.

Due to road transport restrictions, the length of an individual prefabricated pile is limited to about 10 m. Therefore, steel piles have to be supplied in sections and welded together at the site to achieve the designed length. Pre-stressed concrete piles must be manufactured in sections and connected by special joints (Photo 3.4-2).

Steel sheet piles are not available in Bangladesh. As an alternative to imported steel sheet piles, pre-cast concrete sheet piles were designed by the Consultant and fabricated in the country (Photo 3.4-3). They are, however, restricted in length for transportation reasons and can therefore only be used within certain limitations.

Geotextile materials are not being produced in Bangladesh and must therefore be imported. A special composite (sandwich) filter mat of geo-jute from Bangladesh as bottom layer and synthetic geo-textile as an upper layer and a fill of Jamuna sand was manufactured in Germany and tested within a bed protection.

All materials which were imported due to non-availability on the local market - except steel sheet piles and sheet pile anchoring materials - are considered, in general, as reproducible in Bangladesh.

In the following the material procurement in Bangladesh and the imported materials are listed:

<u>Material (Procurement)</u>

- Local:
  - bricks
  - pre-cast reinforced concrete sheet piles
  - pre-stressed spun concrete piles
  - boulders for granular filter, rip-rap
  - concrete (cement, aggregates, reinforcement steel)
  - geo-jute materials
- Imported:
  - tubular steel piles (from France)
  - steel sheet pile material (from France)
  - anchor material for steel and concrete sheet pile cofferdams (from France)
  - structural steel materials (from France)
  - geo-textile filter materials and sand bags/containers (from Germany and France)

Photo 3.4-1: Production of wiremesh at site

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Photo 3.4-2: Pre-stressed spun concrete piles with purpose made joint

Photo 3.4-3: Pre-cast reinforced sheet piles

3.4.3 Availability of Equipment

The inspection of several equipment yards and construction sites of governmental divisions and private companies showed that the availability of ready-for-use special or heavy (floating) equipment was rather limited.

The most important equipment for the construction of the permeable groynes was the equipment required for the installation of the steel and pre-cast concrete piles. The only available 150 ton capacity Manitowoc crawler crane was hired and mounted on a flat barge of 400 ton capacity (Photo 3.4-4). Barge and crane received major upgrading to suit the requirements regarding the projected piling works. In addition, a special hydraulic piling hammer with support crane was hired from a government equipment pool, particularly suited for driving of precast concrete piles.

Photo 3.4-4: Hired crane and barge

In particular the following construction equipment was hired and procured by the Project:

- Local:
  - general construction equipment (trucks, cranes, bull-dozers, graders, excavators, concrete mixers, pumps, survey boat, container camp, etc.)
  - crawler crane on floating barge for pile driving.
- Imported:
  - pile installation gear
  - ancillary equipment (i.e. mooring winches, ropes, rollers, shackles etc.)
  - survey and monitoring equipment

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3.5 CONSTRUCTION OF THE GROYNES

3.5.1 Earth Works

The earth works for the Groyne Test Structure comprised mainly the construction of the impermeable parts of the groynes.

A new main flood embankment, which would be part of the general Gaibandha town protection program was built by BWDB, employing 4 local contractors from the Gaibandha area.

All excavation and filling works in the dry areas were executed by manpower (Photo 3.5-1). Compaction was done by bulldozers or small vibrators. Dredging works for river-based installation of the groyne steel piles were carried out by a crawler crane installed on a barge.

Photo 3.5-1: Excavation by manpower

3.5.2 Revetments and Bed Protections

400,000 CC-blocks with a total volume of 6,500 m^3 were produced at site to serve as slope and toe protection (falling aprons, Photo 3.5-2).

The geo-textile filter mats were installed partly on the dry flood plain and partly under water. In shallow water the unrolling of mats into the water, ballasted continuously with boulders, dumped from a country boat or a flat top barge sufficed (Photo 3.5-3). However, in deeper water the placing of filter mats was carried out by a crane and a purpose-made steel frame.

All revetment protective layers were installed manually (Photo 3.5-4).

Photo 3.5-2: CC-Block production and storage yard

Photo 3.5-3: Groyne G-1, installation of bed protection, dumping of boulders on geo-jute sandwich filter mat

Photo 3.5-4: Installation of rip-rap and brick mattress at groyne G-2

3.5.3 Pile Installation

The tubular steel piles were supplied in sections of 6.5 m to 12 m lengths and welded together at the site to their design lengths. The on-shore piles were installed with a diesel hammer, whereas off-shore piling was done by the Manitowoc crane. The installation of each pile started with a vibration-hammer and was completed by using a hydraulic free fall piling hammer (Photo 3.5-5).

Bored in-situ cast concrete piles were constructed on-shore at groynes G-A, G-B/1 and G-B/2 (Photo 3.5-6). With the exception of two piles which could not be installed to the projected embedment length due to the presence of petrified wood, all bored concrete piles were installed according to the design.

Installation of the locally manufactured pre-cast spun concrete piles (20 m in length) was done by a crawler crane and a hydraulic free fall hammer. Special attention was given to the pile helmet and the piling force to avoid damages to the concrete piles. Additional jetting measures had to be taken to reduce the piling resistance by decreasing the skin friction during the piling process.

Photo 3.5-5: Off-shore installation of steel pile

Photo 3.5-6: Concreting of in-situ bored piles

3.5.4 Sheet Pile Installation

The steel sheet piles for the impermeable groyne section of groyne G-1 were delivered to the site in their final lengths and no further preparation was required. All sheet piles were installed by an electric driven vibration hammer following the "pilgrim method" (Photo 3.5-7).

The locally made concrete sheet piles for the cofferdam of groyne G-A were driven with a diesel hammer. Special attention had to be given to the piling cushion and the selection of the piling force to protect the heads of the sheet piles. All of the parts were installed without major problems.

Photo 3.5-7: Steel sheet pile installation at groyne G-1 with electric vibrator

3.6 ADAPTATION AND EXTENSION OF THE TEST STRUCTURE

3.6.1 Necessity of Adaptation

During the flood season of 1995 the test structure was exposed to strong flow attack which caused local damages at the groynes G-2 and G-3 and at the main embankment between these groynes (as described in Section 3.7). The damages called for immediate repair measures which started in the following dry season. The detailed inspection and assessment of the damages led to the conclusion that the design of the transition between the permeable and impermeable sections of the groynes require substantial improvement.

The impermeable parts of groynes G-2 and G-3 were removed and replaced by steel piles (Fig. 3.6.1) and the main embankment near groyne G-2 was relocated further inland. In addition, the adaptation works comprised the construction of a new groyne (G-A/2) downstream from the initial groyne field to reduce the exposure of the main flood embankment near the Ghagot River mouth.

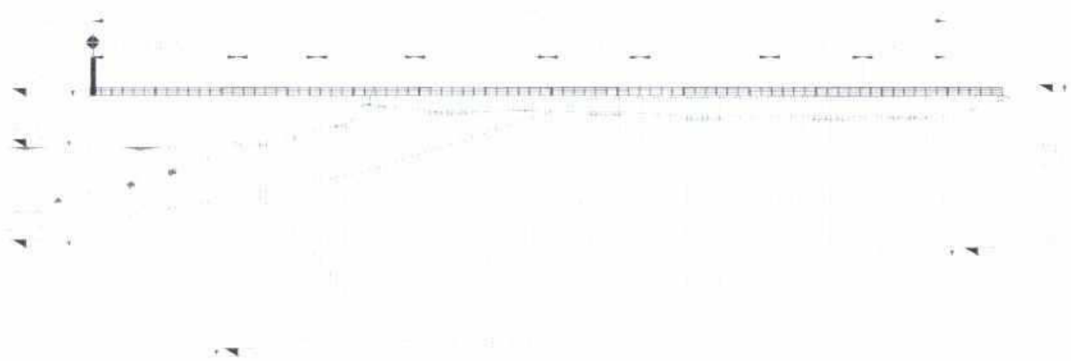

Fig. 3.6-1: Schematical section of permeable groyne (groyne G-2 after adaptation)

The planned modification of the groynes could not be completed within the dry season 1995/96 due to late arrival of required construction materials, hence leaving the completion of work until the next dry season of 1996/97.

3.7 TECHNICAL EVALUATION

3.7.1 Effects on the Local River Morphology

Already during the first year after construction of the test structure, in 1995, a monsoon flood with a peak water level of 22.38 m+PWD (corresponding to a return period of about 15 years) occurred. The groyne field reduced the flow attack considerably and, finally, stopped the extensive bankline erosion (as experienced in the previous years) along the protected area (Fig. 3.7-1).

In 1996, the initiation of a bend cut-off was identified upstream from the test structure between Kundarapara Char and Kharjani Char which developed further in 1998, reducing the discharge of the flow channel directly in front of the groynes (Fig. 3.7-2 and 3.7-3), thus decreasing the exposure of the test structure in the subsequent seasons.

Possible effects of bank protection measures on the morphology of a river are described in general in Annex 1. For highly mobile and dynamic rivers like the Jamuna these effects are particularly difficult to assess. Diverse morphological developments in different scales occur simultaneously causing difficulties in relating the observed morphological changes to individual processes or structural impacts.

Since the groyne field blocked the existing flow channel to some extent, the flow velocity in front of the structure was accelerated. This was followed by (a), a deepening of the channel bed and (b), a local deflection of the river course. Downstream from the structure the flow velocity was decelerated due to the widening of the river cross-section (simplified Bernoulli or momentum approach) and due to energy loss induced by external and internal friction. Moreover, the formation of large eddies due to flow separation at the groyne structure was intensifying local scouring in the structure vicinity. Analogous to the local deflection of the flow direction upstream from the obstacle, the flow vectors downstream from the groyne field pointed towards the bankline which was possibly amplified by mass oscillations (tilting of the water level gradient perpendicular to the river course). Nevertheless, this was also significantly influenced by other pre-conditions.

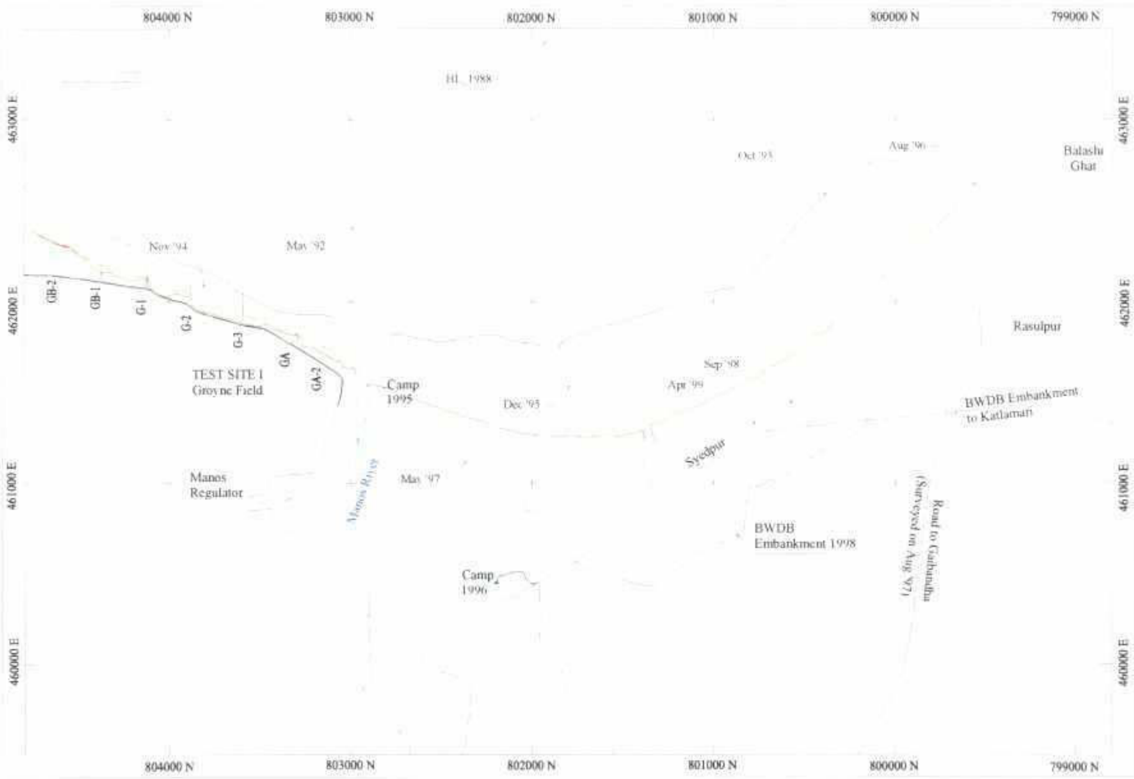

Fig. 3.7-1: Bankline development in the Kamarjani area between 1988 and 1999

Due to the very complex and instationary boundary conditions, a numerical model was set up at the Technical University of Darmstadt, Germany, to investigate the hydraulic and morphodynamic performance (as built) and to allow for an optimization of the structural properties. It has to be stressed that at present, numerical models allow only for approximate results. However, differential examinations are fairly accurate, provided that a proper model calibration is achieved.

The first part of the study was aimed to quantify the morphological effects of the groyne field in the neighbouring area and the structure vicinity. For this reason two test runs (simulating the hydraulic conditions and the morphological changes over a period of one year) were performed, firstly the "as built" case and secondly, the situation without structure implementation ("without" case) as reference.

The results of the model runs are shown in Fig. 3.7-4 ("as built") and 3.7-5 ("without" case). Together with Fig. 3.7-6, showing the differences between the two cases, the following main structure effects are apparent:

- deepening and attraction of the river branch in front of the groyne field
- increase of the bend angle downstream from the groyne field
- prevention of bankline erosion in the structure vicinity

The first aspect, which will be discussed in the following subsection, is related to the structure stability. The two latter aspects were evaluated in terms of positive and negative effects of the protection measure. According to the model results, without the structure, a local bankline erosion of about 200 m/year would have occurred in the projected area which would have increased and developed further downstream in the successive monsoon seasons. Furthermore, the increased angle of the stream bend can be understood as an additional flow resistance in this river branch, contributing to the development of the new channel between Kundarapara Char and Kharjani Char.

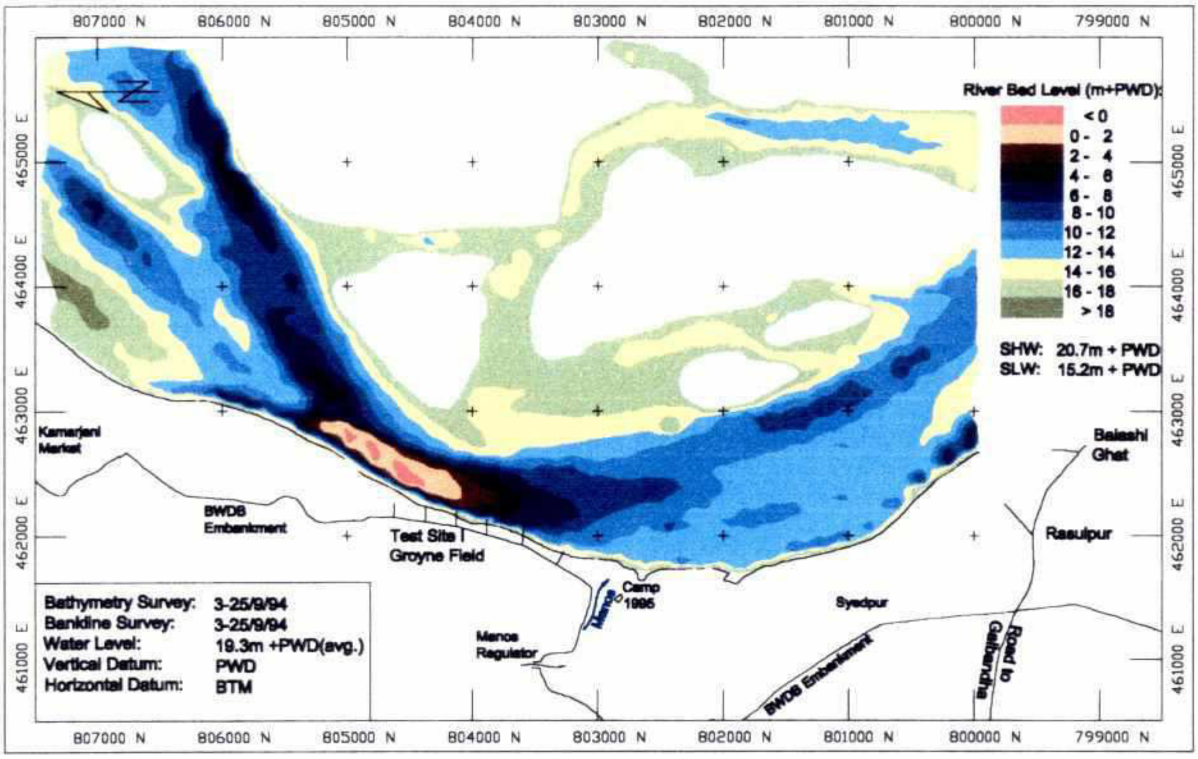

Fig. 3.7-2: Bathymetric situation at Kamarjani in September 1994

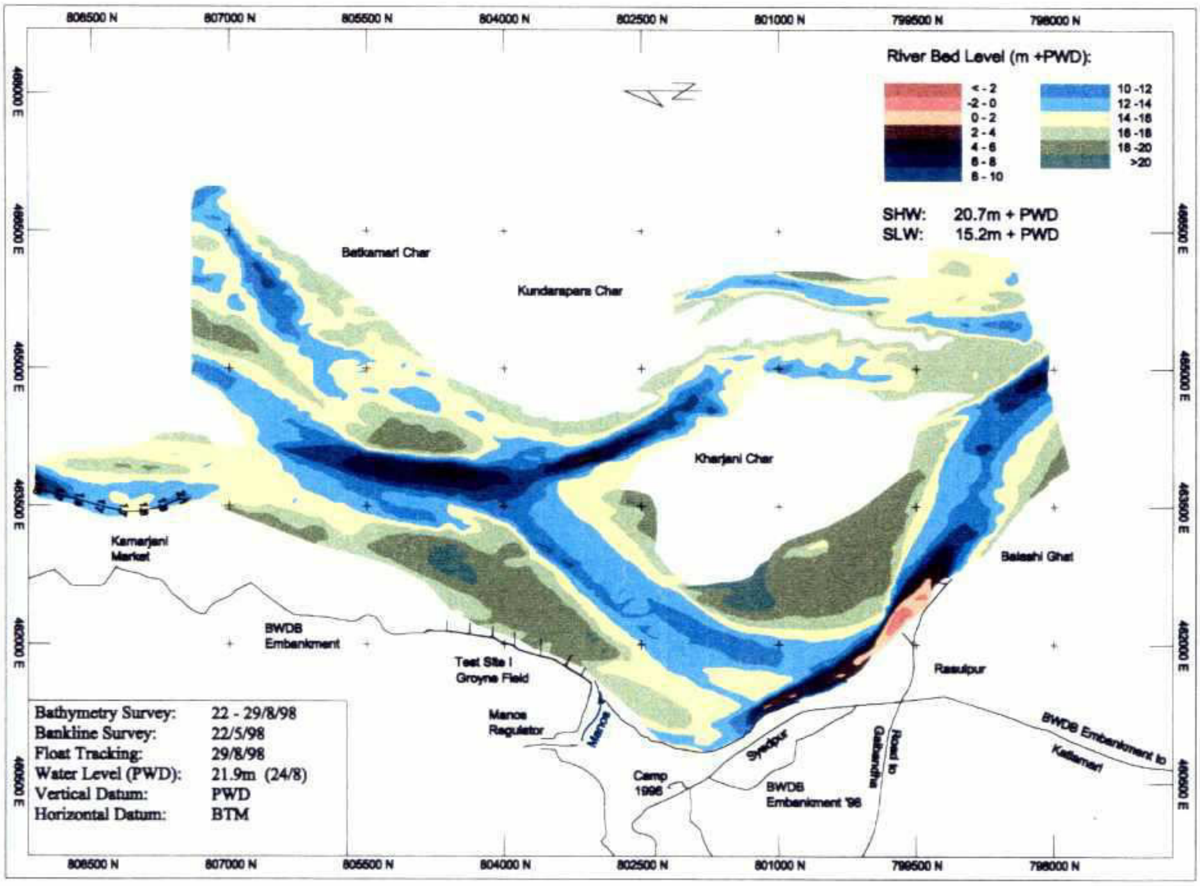

Fig. 3.7-3: Bathymetric situation at Kamarjani in August 1998

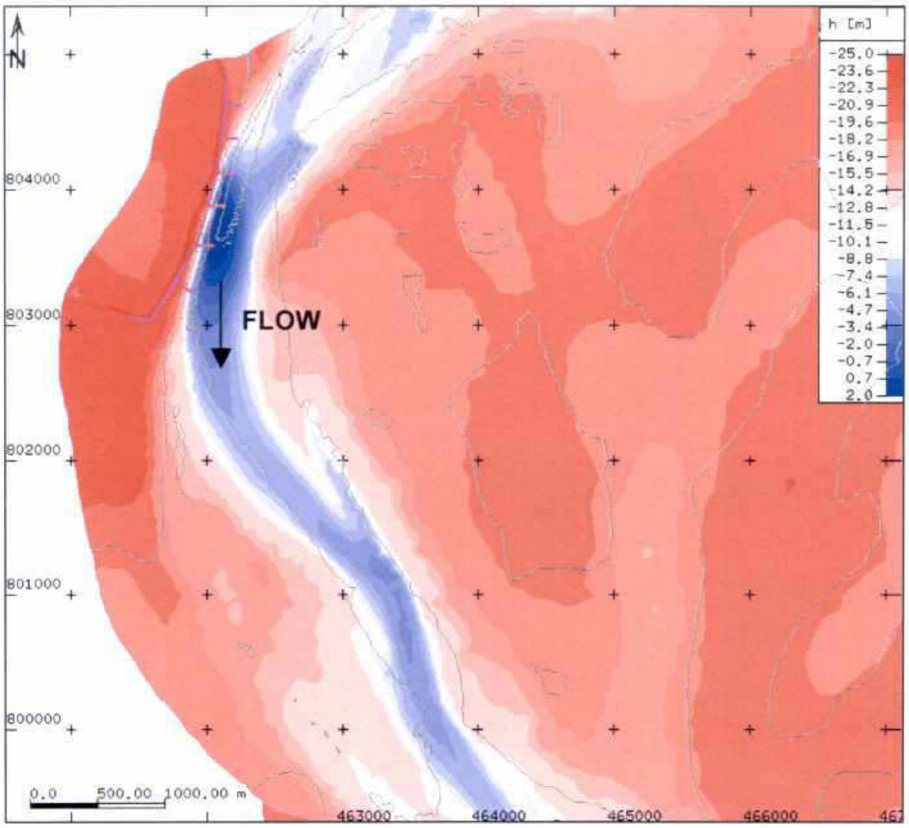

Fig. 3.7-4: Calculated elevations after one year with Groyne Test Structure

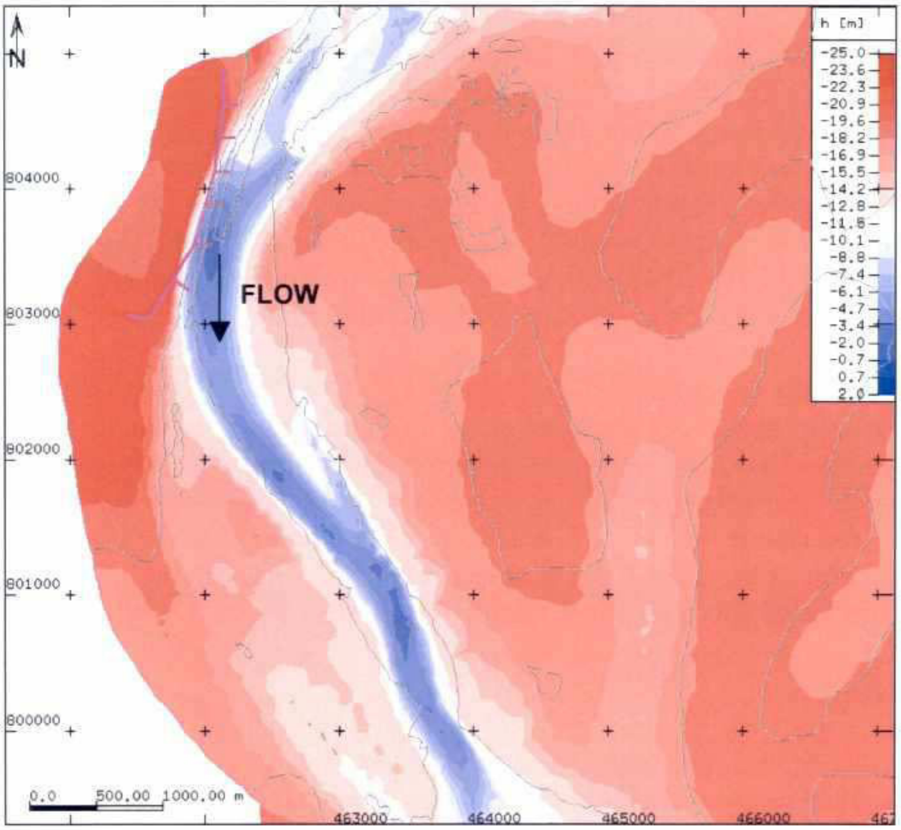

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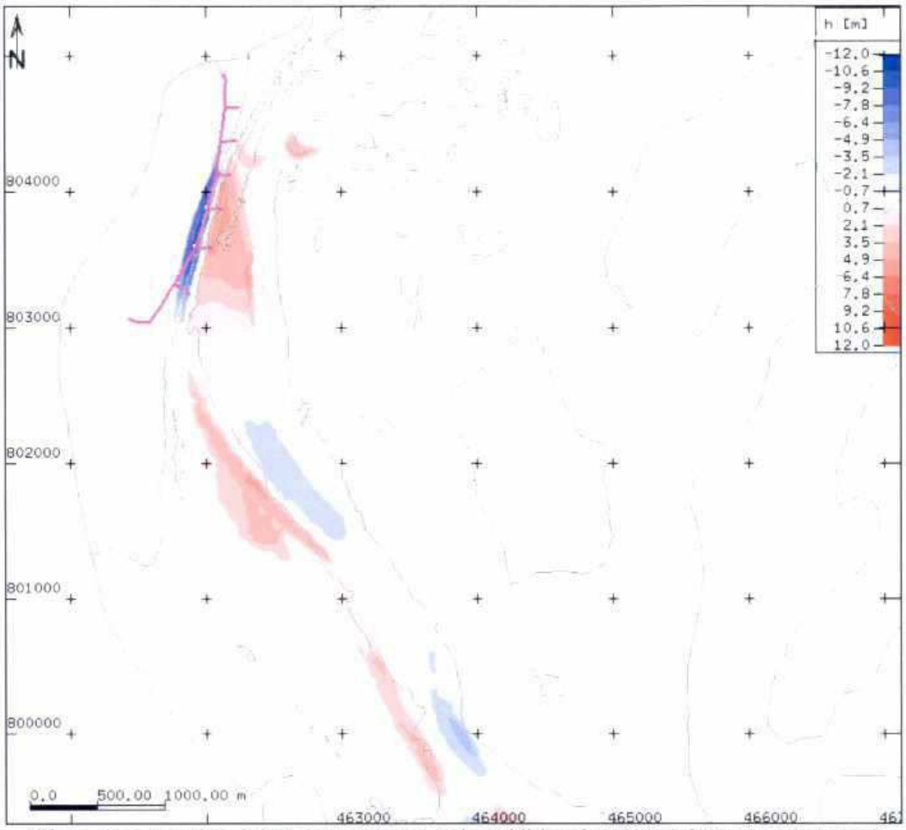
Fig. 3.7-5: Calculated elevations after one year without Groyne Test Structure



(blue = less scouring due to test structure, red = additional scouring due to test structure)

Fig. 3.7-6: Differential plot of situation with and without Groyne Test Structure after one year

3.7.2 Structural Stability

In accordance with the test character of the Project, it was intended to allow for partial damages of structural components in order to identify and define the limits of applied construction elements and to improve the design towards future standard erosion prevention structures. As previously mentioned, damages occurred primarily during the monsoon season 1995. After completion of the adaptation measures in 1997, only minor maintenance works were required. The stability of the different components of the groyne field are briefly described as follows:

(a) Groynes G-B/1 and G-B/2

These supplementary groynes - constructed on the flood plain to control the erosion upstream from the main groyne G-1 - were not subjected to severe hydraulic loads since the centre of attack moved away from groyne G-1 further downstream. Though the bank line approached the piles at the head of groyne G-B/1, the stability was not affected. Only minor erosion of the Durba grass turfing of the impermeable section of groyne G-B/2 was observed. This can be attributed to the late placing of the turf consequently resulting in insufficient rooting.

(b) Groyne G-1

The permeable part of this groyne consists of steel piles with increasing dimensions and spacings towards the river. During the monsoon flood some protective material (rip-rap with D_{50} = 25 cm) was displaced near the head of the impermeable groyne section leading to the local exposure of the underlaying composite geo-jute sand mat. However, the bed protection around the permeable section fulfilled its function satisfactorily and prevented excessive scour around and directly downstream from the piles.

The impermeable part of groyne G-1 was built as a steel sheet pile cofferdam with a toe protection at the river-sided head, consisting of a triple-layer granular filter and a cover layer of rip-rap (D_{50} = 30 cm). Despite these measures, succeeding changes of the riverbed caused some slides to occur at the head of the impermeable groyne section, resulting in a partial failure of the toe protection in this area. However, the stability of the cofferdam was not affected.

(c) Groyne G-2

The impermeable part of groyne G-2 was designed as an earthdam with a rip-rap layer on a granular filter. The permeable part consisted of steel piles. The riverbed around the whole structure was protected by a falling apron of dumped CC-blocks (15 cm to 30 cm in size).

Slope deformation due to scouring at the downstream edge of the impermeable groyne head started with the upcoming flood and led to an initial minor slide of the slope. This was followed by sliding of the cover layer which resulted in a larger slope failure approximately 50 days later. Finally, about half of the impermeable groyne section collapsed into the river (Photo 3.7-1 and Fig. 3.7-7).

The excessive scour development directly downstream of the head of this section can be attributed to the unexpected high return currents within the groyne field. This fact was neither evident nor reproducible in the first physical model tests and was obviously underestimated.

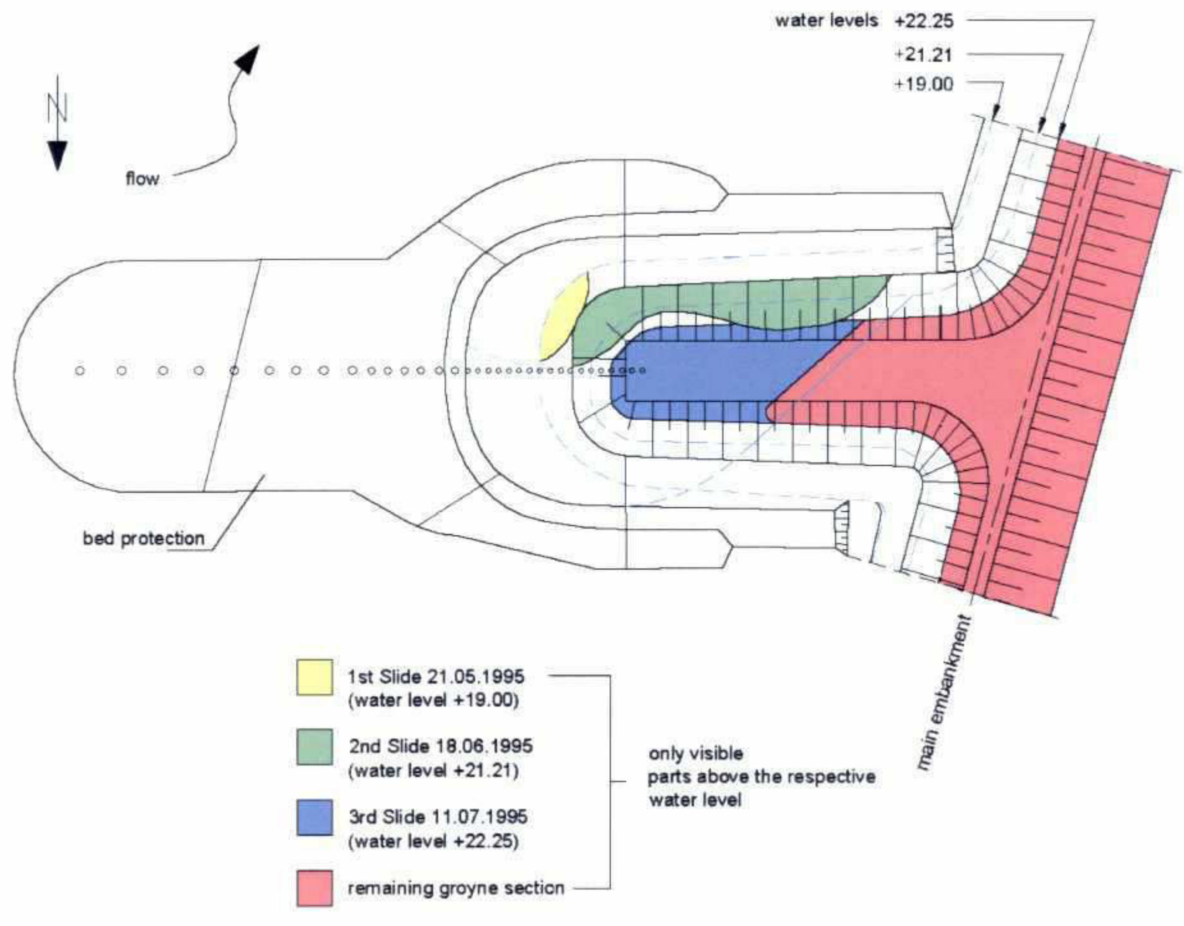


Fig. 3.7-7: Slides that occurred during monsoon flood 1995 at groyne G-2

For executional reasons, the steel piles in the transition zone were staggered in length. With continuing erosion of the river bed at the transition between the permeable and the impermeable groyne sections, the shorter piles became exposed completely but remained fixed to the monitoring gangway and the loads were borne by the adjacent piles. As a consequence, the first four land-ward (longer) piles experienced excessive loads and plastic bending (Photo 3.7-2). As a result, these and some adjacent piles tilted due to insufficient embedded length before the situation stabilized.

Photo 3.7-1: Remnants of groyne G-2 (view from downstream) and erosion of main embankment d/s of groyne G-2

Photo 3.7-2: Recovered tubular steel piles of groyne G-2

Despite the damages in the transition zone, monitoring of the riverbed revealed that the falling aprons showed very good performance and protected the river-sided piles even though the measured scour depth downstream from the head of the permeable section of approx. 8 to 9 m somewhat exceeded the expected order of magnitude (7 m). This excessive scouring might have been initiated by floating debris which frequently covered the whole area upstream from groyne G-2 to a thickness of up to 1.5 m.

(d) Groyne G-3

The impermeable part was constructed as an earthdam, protected by geotextile filter mats covered with rip-rap (D_{50} = 30 cm) at the groyne head and brick mattresses (D = 20 cm) along the trunk. The partial failure of the impermeable part of the groyne developed similarly to G-2, however, with a time-shift of about 6 weeks later. The longer resistance against hydraulic forces could be due to the use of geotextiles instead of granular filters. Geo-textile mats are flexible and ductile to a certain extent, and therefore, capable of increasing the integrity of a protection system.

Downstream from the impermeable part of groyne G-3, the flood plain eroded over an area of about 0.8 ha, with a maximum scour depth of 6 to 7 m located just downstream from the impermeable groyne head. The falling apron around the impermeable groyne head was not sufficient to accommodate a scour of that size and disappeared completely due to the steep downstream slope (Photo 3.7-3).

The upstream side of the impermeable groyne remained intact. This can be substantially attributed to the functioning of the geotextile materials.

The permeable part of the groyne withstood the hydraulic loads undamaged, although no particular protection material was provided around the pile structure.

Photo 3.7-3: Downstream face of eroded impermeable section of groyne G-3

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(e) Groyne G-A

The supplementary groyne G-A was planned to smoothen the transition between the groyne field and the unprotected bankline downstream. The impermeable part was constructed as a cofferdam using reinforced concrete sheet piles with a toe protection of rip-rap on a granular filter. The permeable part was made of pre-cast and in-situ concrete piles built on the floodplain. Bed protection was not provided for these piles.

The floodplain in the groyne vicinity was washed away during the monsoon flood and the permeable part of the groyne became exposed entirely. Though scour depth exceeded the design assumptions around the river-sided tubular steel piles, no damages to the pile structure were observed. To enhance the structure stability, three outer piles were re-driven into the ground by additional 4.5 metres during the following dry season. Some of the more exposed bored in-situ cast concrete piles, for which the embedded length could not be increased after initial construction, were substituted by longer steel piles during the overall adaptation works.

(f) Main Embankment

The river facing slope of the main embankment was protected by Durba grass (between groynes G-1 and G-3) and brick mattressing (downstream from groyne G-2).

Between the groynes G-1 and G-2 minor erosion along the embankment occurred, however, not threatening its stability. Between the groynes G-2 and G-3 the local erosion of the floodplain extended up to the embankment which was undermined and finally failed (Photo 3.7-1). Similarly, the brick mattress of the main embankment downstream from groyne G-3 was undermined by erosion but maintained its integrity and followed the slope of the developed scour (Photo 3.7-4). The anchorage of the brick mattress at the top of the main embankment remained intact.

Photo 3.7-4: Main embankment downstream from groyne G-3 after monsoon flood

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3.7.3 Verification of Hydraulic Design Parameters

Monitoring of currents revealed that the flow velocities at the outer end of the groynes (2.75 m/s) were lower than the design velocities (3.2 m/s). They decreased along the permeable groyne sections to about 1.0 m/s at the transition to the impermeable section. Between the impermeable parts of two adjacent groynes, the reduction of the flow velocities was only minor, due to diffusion from the main channel and due to strong return currents and eddies formed directly downstream from the impermeable groyne heads. The velocities in the large eddies were almost twice as high as compared to the values resulting from the physical model tests. One reason for the strong return currents could be related to the unexpected oblique flow approach.

A comparison between the assumed main design parameters and the corresponding observed values is summarized in Table 3.7-1.

	Unit	Design Value	Observed Maximum Value	Comments
High Water Level (DWH_{25})	m + PWD	22.90	22.38	about 0.5m below design value
Flow velocity	m/s	3.2	≈ 2.75 (near groyne head)	below design value
Flow direction	degrees	0 (parallel)	20 (oblique)	Unpredicted
Return current between the groynes	m/s	0.5	0.8 to 1.0	higher, was not evident from modelling
Critical bed level at groyne head	m + PWD	- 5.0	≈- 5.0	well considered in the pile design
Scour depth related to pile design (below design river bed)	m	4	≈ 6	Underestimated
Maximum scour - depth	m	6 to 7	8 to 9	underestimated
- distance downstream from groyne head:	m	30 to 60	20 to 50	well predicted
Wave height (H_{des})	m	1.0	0.55	obviously conservative assumption
Thickness of floating debris	m	1.0	> 1.5 (estimated)	effects to be further studied

Table 3.7-1: Comparison of design parameters versus observed values

3.7.4 Conclusions

The results of the monitoring and inspection of damages revealed that the scouring processes directly downstream from the transition between the impermeable and permeable groyne sections were underestimated. Consequently, one can conclude that the toe and scour protections at the downstream side of the impermeable groyne heads were undersized and failed due to this reason.

As a result of experience gained with the prototype structures and well supported by the subsequent physical and numerical model tests, the transition between the permeable and impermeable groyne section must be as gentle as possible in order to restrict unfavourable eddy formation and return currents.

It may also be stated that fully permeable groyne structures should be preferred, with the permeability of a groyne structure must be suitably selected. Keeping on the circumstances in mind (e.g. a particular site situation), either a gradually increasing permeability between groyne root and head or a variable permeability along the length of the groyne may be appropriate. In any case, the permeability should range between 40/50% near groyne root and 80% at the groyne head.

Bed protections (groyne G-1) and falling aprons (groyne G-2) around the groyne pile structure have evidently proven their efficiency to secure the pile structure stability. However, it can also be concluded that any such measure should be avoided in view of construction constraints (in terms of time and risk) and high cost. It is more advantageous to slightly increase the piles' embedment length (groyne G-3) and hence allow for more local scour around the piles. This will ensure a more cost-effective, yet safe structure.

Floating debris (water hyacinths) has contributed to the development of additional scour depth. As regular removal of such floating debris appears unrealistic, submerged or gradually submerging groynes definitely reduce the risk of blockage by floating debris.

The tested revetments of the impermeable groyne sections and along the main embankment have proven suitable to withstand the loads of currents and waves. In particular the wire-mesh reinforced brick mattressing has demonstrated its efficiency and adaptability to slope deformation though occasionally suffering from human intervention. Natural slope protection by Durba grass should only be applied for areas with flow velocities not exceeding 0.5 m/s and moderate waves. It must furthermore be considered that the grass sods have to be laid well in time to ensure their adequate rooting before being exposed to the river's flow and waves. The land-sided slopes of the main embankment should also be provided with properly laid and nursed grass sods to prevent surface erosion by rain. Generally a toe protection for the main embankment of at least 5 m width is recommended.

3.8 OVERALL COSTS OF THE GROYNE TEST STRUCTURE

The overall costs for realization of the Kamarjani Test Structure, including land acquisition (borne by GoB), expatriate and local consultancy services, material and equipment procurement, construction works (incl. adaptation measures) as well as monitoring and maintenance works until end of 1999 amounted to about Tk. 595.6 million. A detailed economic evaluation is given in Chapter 6.

4 THE REVETMENT STRUCTURES

4.1 TEST SITE LOCATION

The location finally chosen for the implementation of the first Revetment Test Structure within the project scheme was the river stretch in front of the small village Kulkandi in the Bahadurabad area, situated about four hundred metres south of an important railway ghat at the eastern bankline of the Jamuna. In this area the bankline is forming a head, which is a local obstruction protruding into the peripheral river bed (see Fig. 4.1-1).

The structure was subject to flow attack during the flood of 1997 and which continued in 1998, although the flow channel directly at the structure front lost strength due to a new channel created about 1000 m southwest of the test location.

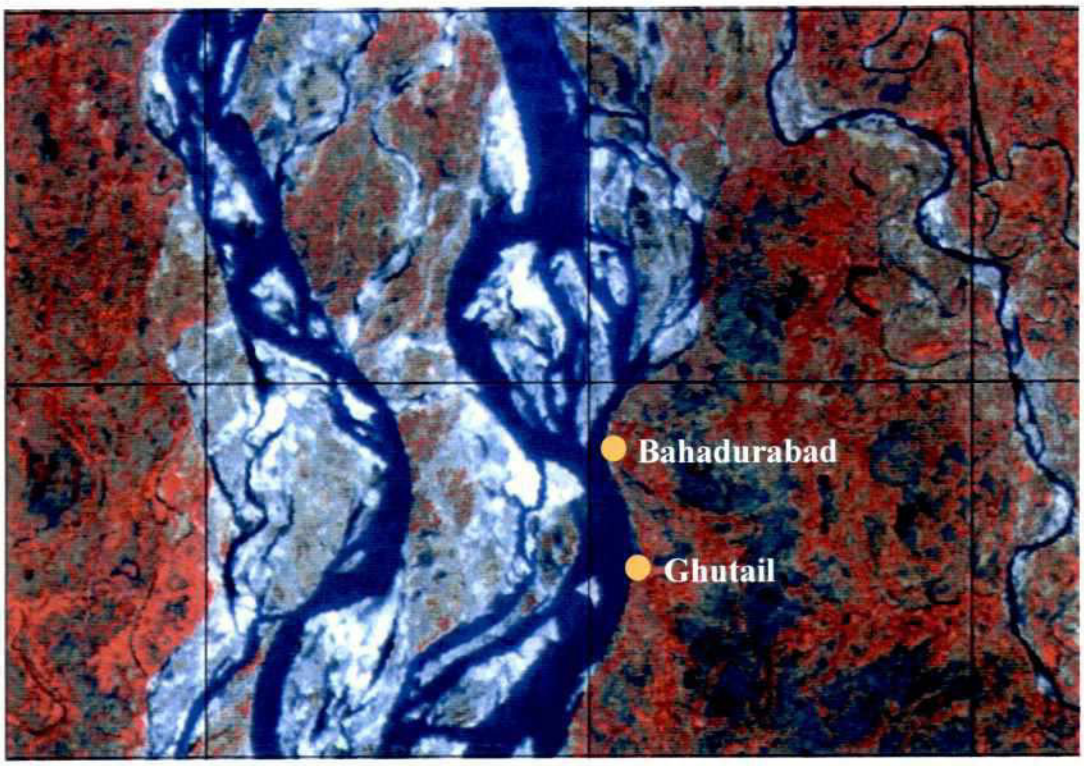

Fig. 4.1-1: Location of Bahadurabad test site at the left bank of the Jamuna river

After successful completion of the two bank protection structures at Kamarjani and Bahadurabad, it was decided to use the remaining funds for the implementation of a third test structure at Ghutail which is located on the left bank of the Jamuna about 4 km downstream from Bahadurabad (Fig. 4.1-1). The design of this test structure was based on the design assumptions and experience made during the planning and implementation of the Bahadurabad Revetment which will be discussed in the following Sections. Specific details related to the Ghutail Revetment Structure are briefly summarized in Section 4.7.

4.2 DESIGN CRITERIA AND BOUNDARY CONDITIONS

4.2.1 Water Levels

A water level frequency analyses at Bahadurabad was carried out on statistical databases, combined with a 1–dimensional flow model which was established by FAP 25. The results of the analysis, covering a 25-year period (1965-1989), are summarized in Table 4.2-1.

Water Level	Reference to PWD	Reference to SLW	Return Period
Standard Low Water (SLW)	13.30 m+PWD	± 0.00 m+SLW	-
Standard High Water (SHW)	18.80 m+PWD	5.50 m+SLW	-
Design High Water Level (DHW)	21.10 m+PWD	7.80 m+SLW	25 years
High Flood Water Level (HFW)	21.40 m+PWD	8.10 m+SLW	100 years

Table 4.2-1: Design water levels at Bahadurabad Test Site

The High Flood Water Level (HFW), representing the statistical highest high water level for a 100-year return period, is the standard design water level for bank protection measures in Bangladesh. For the structures tested within the Bank Protection Pilot Project, however, the so-called Design High Water Level (DHW) was used, corresponding to a 25-years return period. Fig. 4.2-1 shows the band width of water levels recorded throughout the monitoring period (1995-2000). The maximum peak water level (approx. 20.5m+PWD) was measured during the monsoon season of 1995.

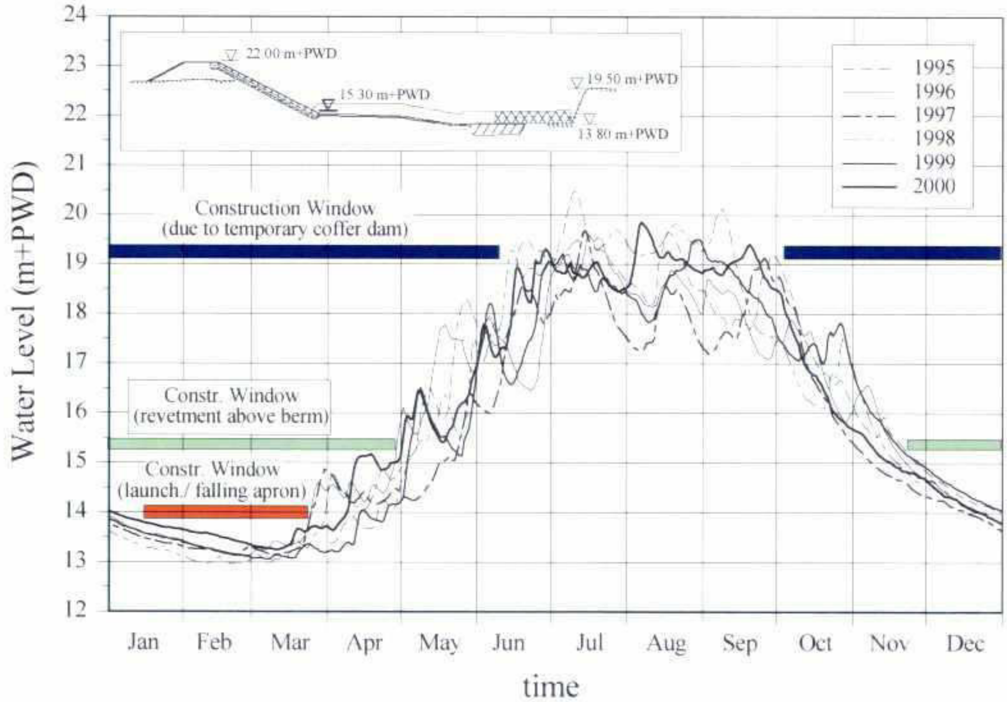

Fig. 4.2-1: Measured water levels at Bahadurabad (1995-2000) and restrictions for the construction window

Besides the implications resulting due to the structure design (crest height, etc.), the time series of water levels - depending on the type of construction – affected the time available for the actual construction of the protective structure. This is shown schematically in Fig. 4.2-1 and for the falling/launching apron but also for the revetment above berm. Additional measures (e.g. coffer dams) might help to extend the time available, nevertheless, certain safety margins have to be considered in terms of the construction window in order to prevent severe consequences for the overall structure resulting from early floods.

4.2.2 Flow Velocity

The depth averaged flow velocity at the revetment toe $\bar{u}_{toe}$ was approximated under idealized assumptions in terms of flow conditions and river bed profile using the TRISULA (two dimensional flow) model. The local design flow velocity $\bar{u}_{des}$ was determined by multiplying $\bar{u}_{toe}$ by a factor governed by the relative horizontal distance between the intersection of revetment and water level (DHW) and the location investigated. The values related to the most unfavourable assumptions in terms of flow approach angle were adapted.

The depth and cross-sectional averaged flow velocity at Bahadurabad was determined at a value of $\bar{u} = 3.5$ m/s (25-year return period). The calculated values of design flow velocities at different locations along an idealized cross-section of the revetments are presented in Table 4.2-2.

Distance from the toe of the revetment slope	Design Depth Averaged Flow Velocity $\bar{u}_{des}$ (m/s)	
	Revetment	Scour Protection
- 30 m	2.8	-
0 m	3.5	3.5
+ 12 m	-	3.6
+ 20 m	-	3.8
+ 30 m	-	3.85

– : towards structure crest
+ : towards river channel

Table 4.2-2: Design flow velocities ($\bar{u}_{des}$) at different locations in front of the revetment structure

4.2.3 Wave Loads

Significant wind generated waves occur at the Jamuna river mainly during the tropical Norwesters and cyclones. The occurrence of tropical cyclones is comparably low in central Bangladesh and may be expected in this region only once in 30 years. Due to this rare recurrence, cyclones have not been considered as a design criterion for the test structures.

Important for the Jamuna river are the squalls during pre-monsoon (Norwesters) and post monsoon periods which generate considerable waves on the river. Squalls are local disturbances causing substantial wind flows with thunderstorms, mainly occurring during the months from March to May, but also at other times of the year. Exceedance probabilities for wind speeds in the Jamuna area were taken from detailed studies carried out under FAP 1. Based on a simple wave prediction model (Bretschneider, Sverdrup, Munk, in Shore Protection Manual, 1984) design wave heights at Bahadurabad Test Site were computed for different return periods (Table 4.2-3).

Return Period (years)	Design Wave Height H (m)	Design Wave Period T (s)
5	0.8	3.0
10	0.9	3.3
25	1.0	3.5
50	1.2	3.8
100	1.3	4.0

Table 4.2-3: Design wave parameters at Bahadurabad site

4.2.4 Vessel Induced Loads

Inland waterway transport does not yet play an important role for the upper reach of the Jamuna river. Therefore, ship induced waves are infrequent and were not given consideration in the design assumptions for the test structures. However, the proximity of the test structure to the railway ferry ghat at Bahadurabad raised the question whether or not additional hydraulic loads and scour induced by ship propulsion could influence on the structure stability. Within the structure's monitoring period, no damages due to propeller erosion have been observed.

4.2.5 Soil Conditions

The geotechnical properties of the test site area are of major importance, particularly in terms of flow resistance, scour development, safety against shear failure, etc. Therefore, in 1992 a total of four exploratory borings with SPT (standard penetration test) and three cone penetrometer tests were carried out between Belgacha and Kulkandi. The nearest boring was located about 2 km south of the final test site.

The subsoil consists mainly of narrow graded silty sand with traces of mica and a content of silt between 9 and 21 % (mean value 13 %). Cohesive layers of clayey sandy silt to sandy silt (CL to ML) were found as cover layers with a thickness up to 5.5 m, but also as about 2 m thick layers at about a depth of 11.5 m below surface. Soil parameters obtained from the various locations and utilised for the filter and revetment designs are given in Table 4.2-4.

As verified by the analysis the soil conditions along the investigated parts of the Jamuna river may be considered as reasonably uniform. Notwithstanding, it is still recommended to evaluate the specific soil properties at any location chosen for future construction works. The ground water table at the time of field investigations was measured between 2.5 m and 3.3 m below surface.

Parameter	Flood Plain Level to 13.30 m+PWD	13.30 m+PWD to 8.30 m+PWD	Below 8.30 m+PWD
Soil classification	Clayey sandy silt to silty sand; (CL – ML)	Fine to medium sand, silty, partly clayey (SM)	Sand
Grain size distribution (mm) d_{60} d_{50} d_{10}	 0.03 to 0.09 0.02 to 0.07 0.003 to 0.03	 0.11 to 0.20 0.09 to 0.22 0.04 to 0.07	 0.20 to 0.26 0.15 to 0.22 0.06 to 0.08
Coefficient of uniformity $U = \frac{d_{60}}{d_{10}}$	3 to 10	3 to 4	3 to 4
Coefficient of permeability (m/s)	-	$3 \cdot 10^{-5}$	
Angle of internal friction φ' (°)	25 to 27.5	27.5 to 32.5	
Cohesion c' (kN/m²)	7 to 20		0
Unit weight/submerged unit weight γ/γ' (kN/m³)	18/8		

Table 4.2-4: General soil properties at Bahadurabad area

4.2.6 River Bed Profile

Due to rapid bankline changes of the Jamuna river, it was important to predict the morphological situation within the implementation period for the planned site location. The different likely profile developments resulting from hydraulic model tests are summarized in Fig. 4.2-2.

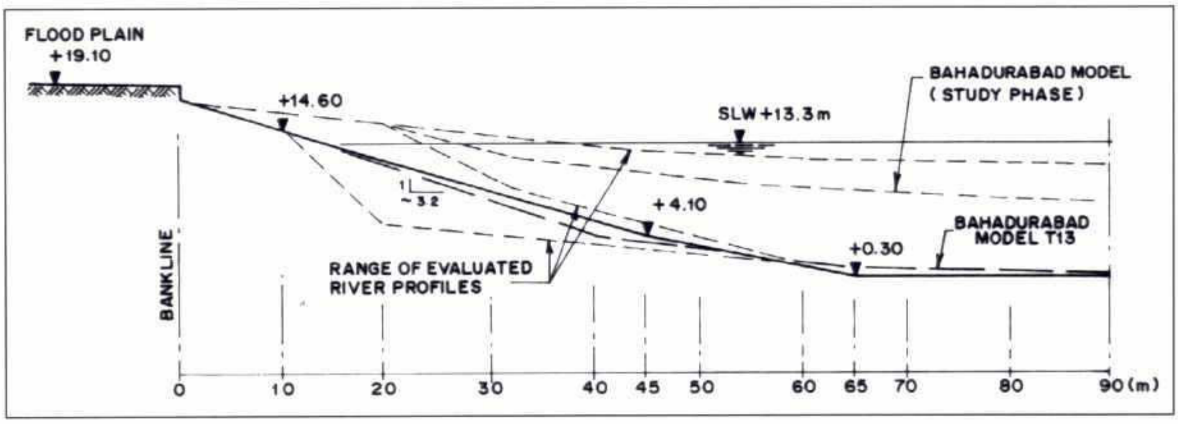

Fig. 4.2-1: Estimated development of the river bed profile

4.2.7 Design Methods

In general the design methods as well as theoretical and empirical formulae used meet the state of the art solutions. Therefore, these methods are not described here, but can be found in the Design Manual for Bank Protection Structures.

4.3 LAYOUT DESIGN OF THE PILOT STRUCTURE

4.3.1 General Structure Layout

The first step in the preliminary design of a revetment structure is to identify the plan view (footprint), taking into consideration the overall performance and the overall stability of the structure. Revetment structures have to be linked to the main embankment to avoid any flow channel behind the structure, since the inner slope is normally not protected sufficiently against erosion. This will also have to be taken into account in the design of the upstream termination. The test structure was aligned at a slight angle to the assumed river flow, to allow for a more or less uniform flow along the individual revetment test sections (see Section 4.3). The developed length of the revetment structure is 780 m (crest line) protecting a bankline of approx. 680 m.

The original design of the test structure was evaluated on the basis of hydraulic model tests. The general layout (alignment, up- and downstream termination) and the related flow pattern in front of the structure as well as the location and extent of scour holes was investigated in smaller scale (1 in 60, at Faridpur, Bangladesh). More detailed models (1 in 15 and 1 in 25) were applied for studying the behaviour of the falling aprons (at Chanaz, France).

According to the results of the physical model tests the deepest scour was expected to develop near the upstream termination of the test structure. The estimated scour development, together with the preliminary plan view of the Revetment Test Structure, is shown in Fig. 4.3-1.

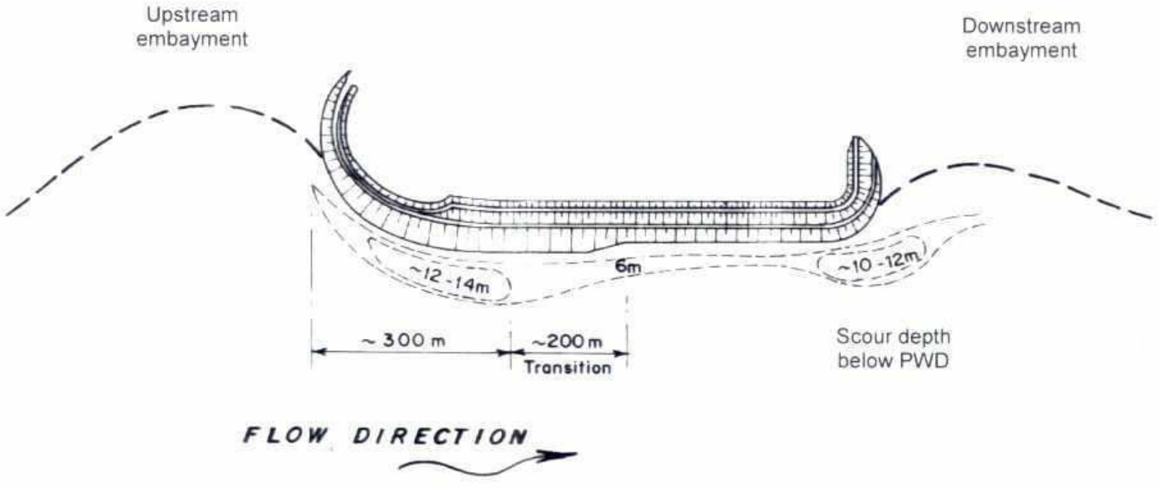

Fig. 4.3-1: Expected scour depth along the structure front

4.3.2 Upstream and Downstream Terminations

The results of the physical model tests demonstrated the necessity of applying a sufficient radius for the upstream termination to reduce the risk of eddy formation and thus the expected scour depth. By increasing the radius from 50 to 450 m, the expected scour depth along the structure decreased from 20.5 m to 13 m. It was anticipated that the applied configuration would withstand an embayment depth of about 250 m at the upstream river bank. At present, one may deduce that an embayment should be considered in the dimensioning of the upstream termination, in order to prevent from damages, endangering the overall integrity of the structure.

Since the downstream termination is of smaller importance for the structures overall stability it was designed with a radius of 50 m. However, the length perpendicular to the main crest line must be sufficient to consider a potential embayment downstream from the structure.

To prevent the landward side of the revetment from erosion attack in case of high floods and severe upstream erosion, a connection between the revetment and the main flood embankment is very important. This connecting dyke or embankment, protected either by grass or brick mattresses was to be built under BWDB responsibility but was not yet been completed before the monsoon period 2001.

4.3.3 Modifications of the Initial Design

The beginning of the construction works in the dry season 1995/96 was delayed due to difficulties in the land acquisition, problems regarding material supply of rocks and stones as well as to matters related to mobile equipment for special construction techniques. After the initial start of works it became clear that the locally available construction capabilities could not meet the requirements for construction of underwater slopes. In order to comply with the project strategy (to plan and implement structures suitable for local construction capacity), it was decided to place the falling apron in almost horizontal alignment on the dry flood plain at the toe of the revetments, allowing the construction in a dry construction pit just above SLW. Due to the delays, the remaining time within the construction window 1995/96 was neither sufficient to complete the structure nor to at least even allow for a stable condition to resist any potential flow attack. As a result, substantial parts of the already completed earthen substructure were washed away during the following monsoon season.

The structure was completed within the succeeding dry period 1996/97. Due to the restricted area of land available for the second construction site, the radius of the upstream termination had to be reduced (two segments with r = 200 m and 75 m, respectively). No damages associated to these modifications occurred within the project period.

The modification of the structure design had to be carried out taking into consideration of the already procured construction materials, such as geotextile filter materials, geotextile mattresses, boulder sizes and concrete block sizes. Consequently, certain compromises had to be accepted for the modified design. In general, it was concluded that the toe protection (the so called launching and falling apron) would follow the developing underwater slope and finally remain stable in a position similar to the original design slope (Fig. 4.3-2).

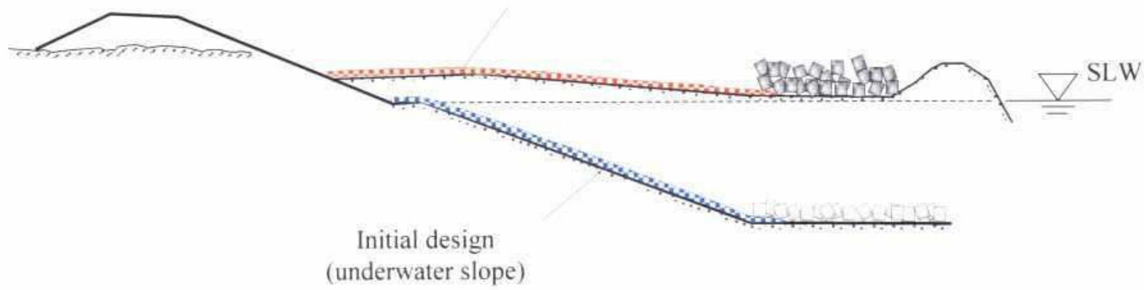

Fig. 4.3-2: Original design and modified cross section for the revetment structure (sketch)

The plan view of the Revetment Test Structure at Bahadurabad (final design) illustrates the general ("as built") layout of the structure, including the modified upstream termination and various test sections (Fig. 4.3-3). Each test cross-section consists of different structure components (earthdam with protective layers, toe protection, launching and falling apron), which are described in Subsections 4.3.4 and 4.3.5.

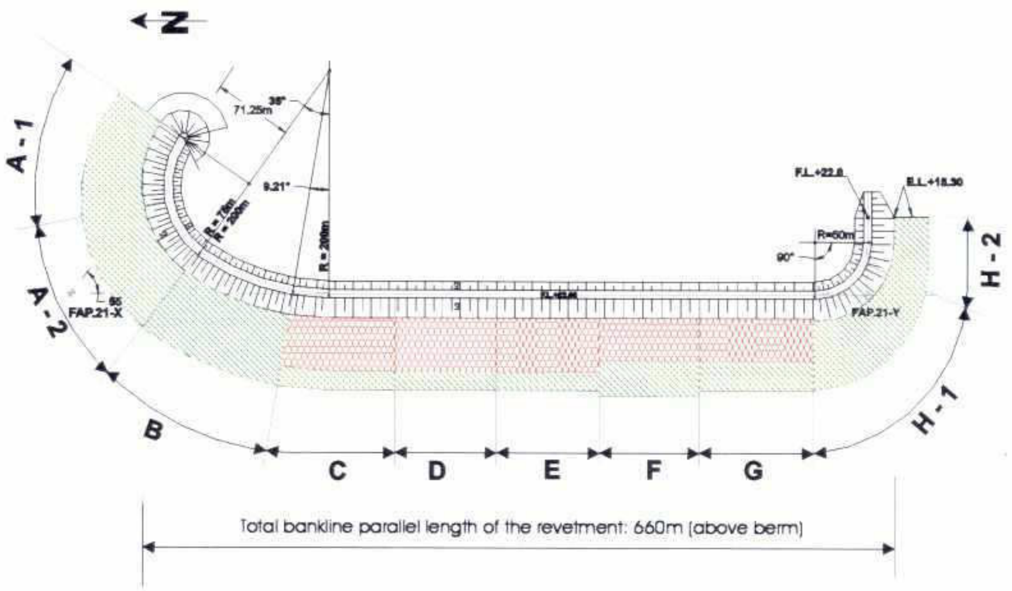

Fig. 4.3-3: General layout (plan view) and test sections (A-H) of the Bahadurabad Revetment Test Structure

4.3.4 Cross Sections – Variation of Filter and Armour Layers

One of the primary objectives of the actual testing of the revetments was the performance and stability of different structure components taking into account various materials used as armour and filter layers. Over the total length of the revetments at Bahadurabad, 6 different sections (with a length varying between 80 – 100 m each) plus test sections at up and downstream termination were implemented. The layout of the different cross sections are depicted in Fig. 4.3-4.

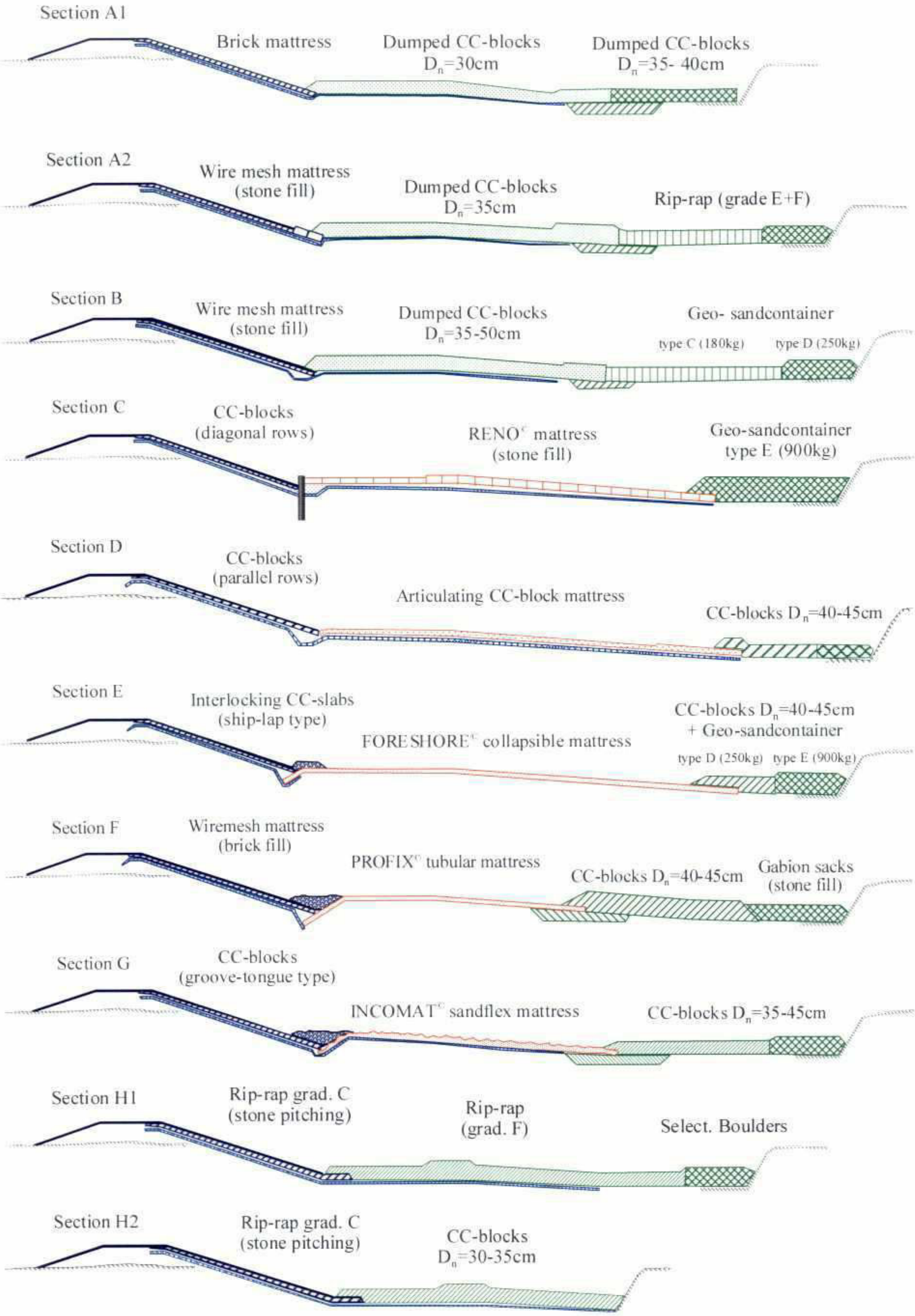

Fig. 4.3-4: Various test sections considering different filter and cover layers at Bahadurabad

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4.3.5 Structure Components and Materials

4.3.5.1 Revetment (above berm)

The design crest level of the revetment structure, assumed to ensure negligible overtopping rates, was calculated at 22 m+PWD based on hydraulic boundary conditions for a return period of 25-years.

The crest width was set at 7.5 m, thus enabling its use as a stockpile area for boulders which required immediate availability in case of any emergency measures during the monsoon season. The slope of the revetment (above berm) was chosen to 1V : 3H, in order to take into account local variations of the subsoil properties. The crest surface was reinforced by a compacted khoa layer. The armour layer protecting the outer slope of the earth dam (revetment substructure) consisted of different element types, chosen for their reproducibility in Bangladesh and ease of installation in areas located above water level. All of the following alternatives were designed for installation on geotextile filter mats and granular filters respectively (compare with Fig. 4.3-4):

- Brick mattress (traditional system of two brick layers, the top layer placed in fish bone pattern, between galvanised steel wire mesh); mattress thickness 15 cm and 20 cm;
- RENO©-mattress (imported), 36 cm thickness, filled with full-size bricks; placed on top of a 25 cm thick intermediate granular layer and a geotextile filter mat;
- wire mesh mattress (local manufacture), 23 cm thickness, stone filled (D_{50} = 15 cm, grading range B), with a 25 cm thick intermediate rubble layer placed on the geotextile filter;
- rip-rap of hard rock (D_{50} = 25 cm, grading range D), with colloidal cement grouting;
- cement concrete blocks (D_n = 30 cm), chipped bricks (khoa) as coarse aggregate, single layer, hand-laid in rows parallel to the berm, with staggered joints;
- interlocking cement-concrete units, factory made (locally) tongue-and-groove type, thickness 15 cm;
- cement concrete blocks, chipped bricks (khoa) as coarse aggregate single layer, hand-laid in rows diagonal (at 30°), placed on a two-layer granular filter or a two-layer khoa filter, respectively;
- wire mesh mattress (local manufacture), 36 cm thickness, filled with full-size bricks, placed directly on the geotextile filter mat, without intermediate granular filter;
- interlocking cement-concrete blocks, factory made (locally), ship-lap-type, thickness 17 cm, and
- rip-rap of hard rock (D_{50} = 20 cm, grading range C), with bitumen grout (full-depth pattern grouting), layer thickness 40 cm and 50 cm.

4.3.5.2 Transition between Revetment Slope and Launching Apron

To increase the stability of the critical transition between the outer slope of the revetment and the launching apron, it was decided to place additional concrete blocks or rip rap, respectively, at several test sections (E – G, see Fig. 4.3-4).

4.3.5.3 Launching Apron

A launching apron consists of an integrated and articulating mattress system (like sand- or concrete-filled geo-synthetics) or of interconnected concrete blocks, which are covering the slope in case a scour hole is created at the structure front. The launching aprons were placed almost horizontally on the dry flood plain in front of the revetment to protect the dam structure from erosion in case of severe scouring. Since the aprons were designed to adjust to potential scour holes, the mattress systems and the interconnecting elements used have to cope with tensile stresses. For this reason, concrete block elements, which were mostly placed on geotextile filter mats, were linked by interlocking cables, to act as a flexible and ductile "apron", increasing the stability of the river bed profile. Two further types used as launching apron were employed: Firstly, stone filled wire mesh mattresses and, secondly, sand filled geotextile elements (tubes, bags, etc.). To control potential tension forces exceeding the

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frictional resistance between apron and subsoil, the aprons were anchored at the toe of the revetment structure. The following system components were tested:

- RENO©-mattress (imported), 23 cm thickness, stone filled (D_{50} = 15 cm, grading range B), with a 25-cm thick intermediate rubble layer placed on the geotextile filter;
- mattress system of tubular tailored, woven fabrics (PROFIX-mattress©), sand containers filled with selected Jamuna sand in dry condition;
- collapsible block mattress of woven fabrics with filter points (INCOMAT-Sand Flex©) filled by selected Jamuna sand;
- Collapsible block mattress of woven fabrics (FORESHORE©), filled by cement-sand grout;
- Articulated CC-Block Mattress System, concrete blocks cast directly on the geotextile filter mat, connected by a system of parallel and perpendicular steel wire cables, and
- Articulated RENO-Mattress©, gabion-type wire-mesh boxes interconnected by steel wire cables.

4.3.5.4 <u>Falling Apron</u>

The falling aprons were made of single gravity elements, like concrete blocks, boulders and geotextile sand containers, placed at the river-sided end of the launching apron. They are designed to be stable during a current attack but are intended to proceed down a scour hole (or an underwater slope) after successive erosion and thus to stabilize the scour profile in order to prevent the structure from undermining and from base failure. The amount of material stored in the falling apron was designed to cover the slope of an estimated potential scour hole by a double layer of blocks/sand bags used. Modules used for the falling apron are as listed below:

- Cement-concrete blocks of sizes ranging from D_n = 25 cm to D_n = 45 cm;
- stones and boulders, D_{50} = 30 cm and D_{50} = 35 cm respectively, with a range of D = 25 cm to 45 cm (W = 40 kg to 230 kg);
- gabion sacks, filled with stones D_{50} = 15 cm, grading range B; fill volume about 0.65 m^3, weight approx. 1,300 kg/sack;
- geotextile sand containers of different fill volume, ranging from 180 kg/bag to 900 kg/bag, and
- scour protection mattress (FORESHORE© collapsible block mattress).

4.4 CONSTRUCTION OF THE REVETMENTS

4.4.1 Earth Works

The embankment was built mostly using soil excavated which profiling the flood plain for the aprons at the toe of the revetment. Due to difficulties experienced during the first construction phase while earthworks were mainly carried out by labourers (Photo 4.4-1), it was decided to use mechanical equipment for the earthworks in the second construction phase (e.g., bulldozers, excavators, pay-loaders and trucks) to allow better progress. The preparation of the subsoil (profiling and compaction) at the apron area was done by bulldozers and trucks. In some areas the existing soil properties were rather bad (e.g., clay lenses) hence these had to be replaced by better material of sufficient quality.

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Photo 4.4-1: Earthworks carried out mainly by labour forces during the first project phase

4.4.2 Revetments (above berm level)

4.4.2.1 Filter Layer

After preparation of the revetment substructure, it was initially planned to spread out the geotextile mats at their final location and to sew the sheets by special hand held sewing machines. Therefore, due to the lack of skilled workers and the frequent breakdown of the sewing machines, it was decided to ensure the stability of the subsoil by individual overlapping mats instead. The placing of geo-textile or granular filters did not cause any further technical problems.

4.4.2.2 Cover Layer

Brick mattresses are a rather common construction material in Bangladesh and were applied as cover layer for several parts of the revetment (Photo 4.4-2). However, the availability of high quality bricks is sometimes insufficient and brick production is more or less limited to the dry season. Due to the large amount of bricks needed and to prevent delays regarding the start of construction, the material should be ordered at least one year in advance. The locally or on site manufactured wire mesh used for brick (or stone) mattresses was not durable enough because of imperfect galvanization.

One test section of the main embankment was protected with ship-lap type factory-made cc-slabs (element thickness 15 cm). Due to interlocking, these light protection units can withstand strong hydraulic loads and appear to be a very economical solution. Nevertheless, within test Section E the subsoil migrated beneath the cover layer even though a special composite geotextile filter (Type GF-5, non woven needle punched and coarse fibre layer) was utilised. This was mainly related to insufficient and non-uniform surface pressure on the geotextile filter due to the lightweight of the slabs as well as to manufacturing tolerances. Besides this, the factory-made slabs suffered considerably from transportation damages. Under the prevalent conditions it has to be concluded that ship-lap type units should not be considered as a viable solution.

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The crest of the embankment was finished with a compacted layer of a khoa-sand mixture. The inner slope was covered with grass sods on geo-jute soil saver.

Photo 4.4-2: Slope protection by brick mattresses (test Section F, view to the north)

4.4.3 Launching Aprons (revetments below berm level)

Installation of launching aprons has not generally been a technical problem but due to the large quantities of material to be installed above water level (dry conditions), this structure component was always critical since it had to be completed before the water level in the Jamuna river started raising.

When Reno© or other wire mesh mattress are used, special attention must be paid to the maximum filling level and to a proper fastening of the wire mesh top. Furthermore, a sufficient tightening of the anchor cables (eventually by minor pre-stressing) is required. Placing of gravel filters and filling of the wire mesh mattress (gabions) was done manually (Photo 4.4-3).

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Photo 4.4-3: Reno©-mattress placed on geo-textile (stone filled cages to be closed by wire mesh)

For the implementation of articulating CC-block mattresses (Section D), the in-situ cast concrete blocks were anchored by steel needles in the geotextile material. The steel needles as well as the anchor cables were installed prior to concreting to allow for a good connection between filter mat and articulating CC-blocks (Photo 4.4-4). The installation of the articulated CC-block mattress was time consuming but caused no specific problems.

Photo 4.4-4: Casting of articulating CC-blocks, interconnected by steel cables

Placing of the Foreshore© collapsible block mattresses (Section E) and inserting the slope cables was carried out by hand without problems. Difficulties occurred when the mattresses had to be filled with concrete (grouting) using a special colcrete mixer pump to achieve the design filling level (d=25 cm). The matter was solved by using additional filling holes to reduce the filling length and additional pressure by "foot stamping" (Photo 4.4-5).

Phòto 4.4-5: Filling of the Foreshore© collapsible mattress with cement grout

The Profix© tubular mattresses (Section F) were partly filled by coarse sand and partly by sand bitumen, using a steel pipe inserted at distances of 6 m through a hole into the tubes. The overlapping width between two adjacent mattresses was fixed at 0.70 m (total width of single mattress: 6m). The installation of Profix© tubular mattress caused no problems.

The Incomat© sand-flex mattresses are formed by 80 by 120 cm sized geo-textile cushions. Each 5m wide mattress has 8 cushions. Filling the mattresses with sand using the hydraulic method failed, therefore, a new filling method using air pressure was applied and led to a significantly shorter construction time.

4.4.4 Falling Aprons

Geo-textile sand containers used as falling aprons (Section B, C, E) were partly supplied ready-made and partly sewed from geo-textile sheets on site. The containers were filled with sand/earth close to their final position. Smaller containers were dumped manually, whereas the larger containers were transported by front-loaders. The largest containers (weight of 900 kg) were filled directly 'in situ'.

The ready-made supplied gabion (wire mesh) sacks (Section F) were filled with stones/boulders (D_{50} = 15 cm) to a final size of approx. 0.65 m^3 (1300 kg) and dumped by a front-loader.

Concrete blocks of all sizes between 30 to 50 cm were transported from the storage area to the construction site by e.g., rickshaw-vans, tractors or trucks. Depending on the design of the falling

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apron, the cc-blocks were dumped randomly (Photo 4.4-6) or placed in the designed mode, respectively.

Photo 4.4-6: CC-blocks and gabion sacks (to the left) used as falling aprons

4.5 FUNCTIONAL EVALUATION

4.5.1 Overall Morphological Changes

The morphology of the Jamuna river is extremely sensitive to any disturbances due to the very fine and thus highly mobile bed material. Consequently, the Revetment Test Structure may change the river morphology several kilometres downstream. However, due to the lack of comprehensive spatial and temporal series of river bathymetries (and planform records) before and after completion of the test structure, it is impossible to carry out statistical analysis on structure induced bank erosion which would be required to quantify this effect. Nevertheless, keeping the fact in mind that the Revetment Test Structure at Bahadurabad does not induce large-scale backwater effects and does not significantly reduce the active width of the river, the overall morphological changes are expected to be rather small.

In this context, it has to be clarified whether an erosion of the bankline near Belgacha would have been likely even without the Revetment Test Structure, as illustrated schematically in Fig. 4.5-1. This aspect has already been discussed in connection with the numerical model calculation for the Kamarjani Test Site (refer to Chapter 3).

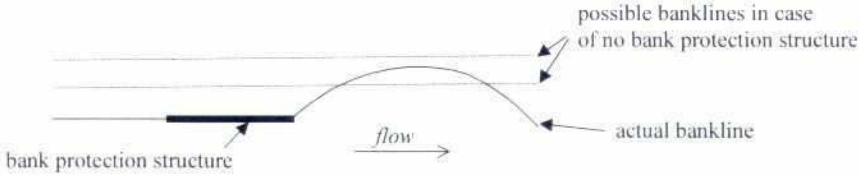

Fig. 4.5-1: Embayment formation downstream from the test structure and effect of impeded bank erosion at test site (schematically)

The primary effect of the revetment has been the successful stopping of bank erosion in the protected area. The bankline erosion rates in the vicinity of the test structure reached values between 100 to 400 m/year during the monsoon periods 1994 to 1996, whereas in 1997 and 1998 the erosion rates decreased to about 20 to 200 m/year. The bankline development between Bahadurabad Ghat and Ghutail during the period 1989 to 1997 (Fig. 4.5-2) accentuates the structure performance, clearly showing the protected area behind the revetment.

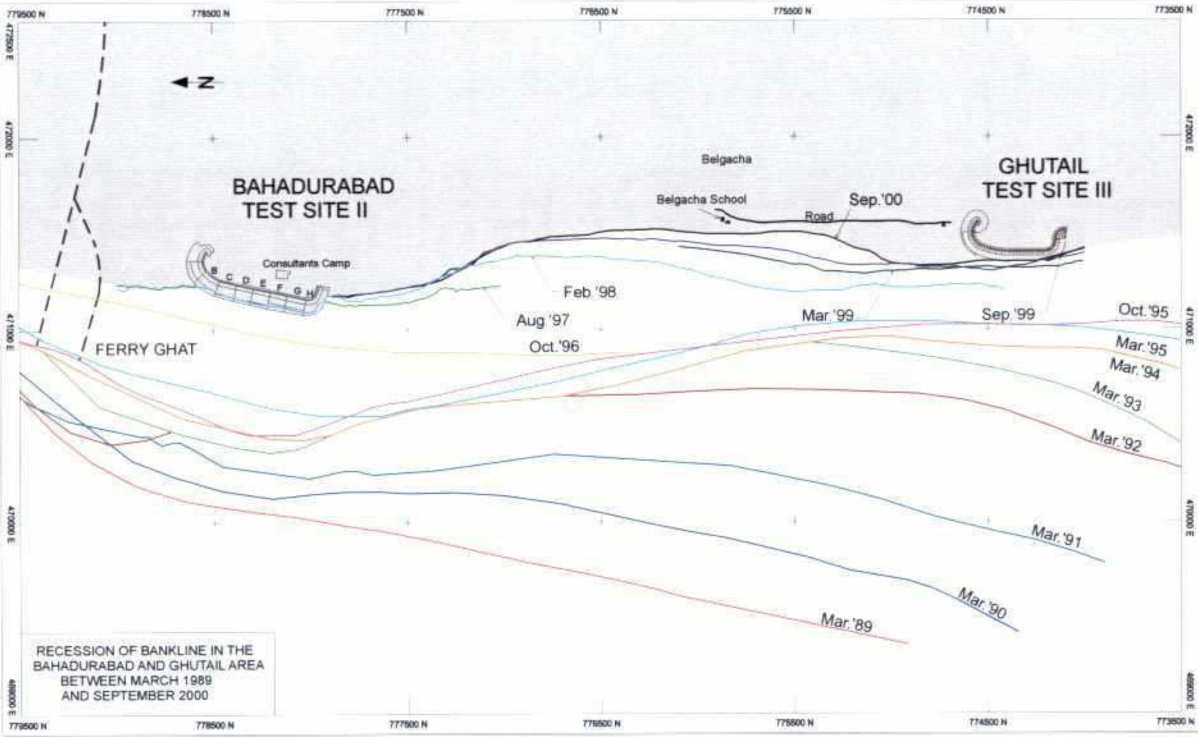

Fig. 4.5-2: Bankline development in the Bahadurabad and Ghutail area between 1989 and 2000

4.5.2 Morphological Changes in the Test Site Area

In spring 1997 the bankline (defined as the edge of the flood plain) in front of the Revetment Test Structure started to erode. The natural cofferdam protecting the revetment works during construction was still existent until mid June but after its breach on June 20 the test structure came under flow attack. The very rapid bankline recession in front of the Test Structure (erosion rates up to 2 m/day) is demonstrated in Fig. 4.5-3 and 4.5-4.

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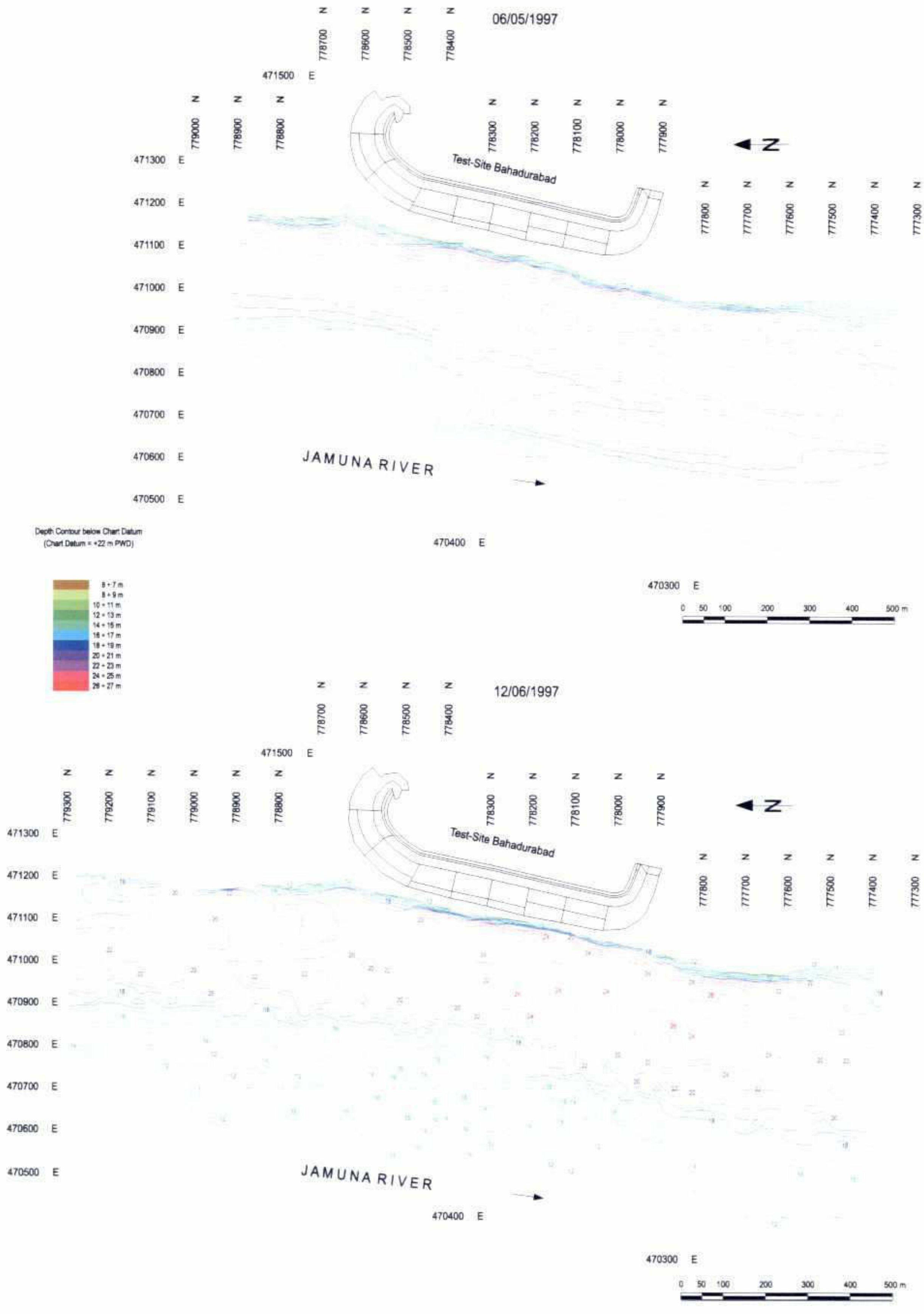

Fig. 4.5-3: Morphological situation in the vicinity of the Revetment Structure on May 06 (above) and June 12, 1997 (below)

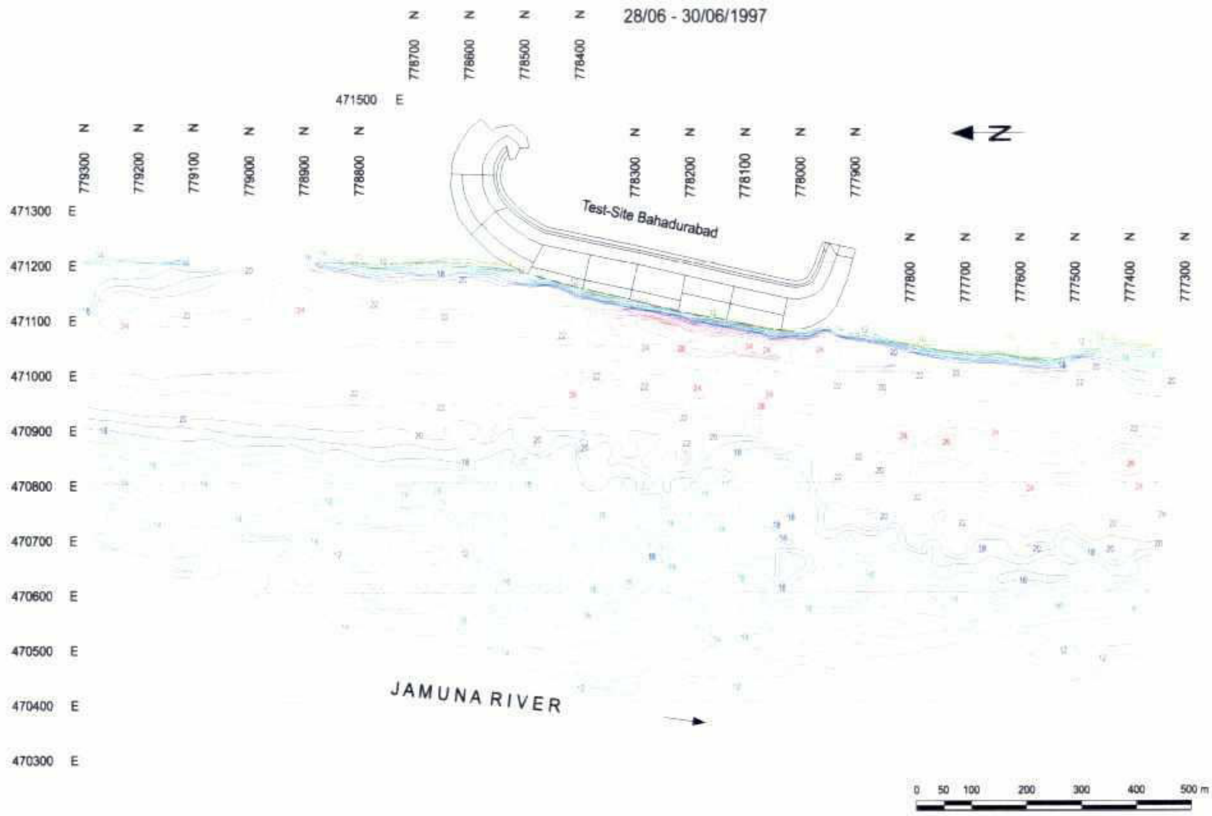

Fig. 4.5-4: Morphological situation in the vicinity of the Revetment Structure on June 30, 1997

Within approximately three weeks after the bankline reached the falling apron of the revetment, substantial bank erosion occurred closely downstream from the structure (near to Section H). After June, the bank line immediately upstream and downstream from the test structure became stable and the main location of bank erosion shifted about one kilometre in downstream direction. There, in the vicinity of Belgacha, a 2 km long embayment was created, more or less, to an extent as foreseen in the design.

4.5.3 Local Scour

Deepening of the river bed in front of the revetment was generated by transversal shifting of the main channel (course) and was amplified by scouring induced by the local structure. With increasing bank erosion, the structure functioned as an obstacle, and by the end of the 1997 monsoon season, the downstream edge of the falling apron protruded about 80 m into the river channel. This resulted in a so-called protrusion scour which remained relatively stable in depth and position.

The main scour hole (depth more than 4 m below PWD) extended over 800 m parallel to the bankline, starting at Section D to approx. 400 m downstream from the Test Structure (Fig. 4.5-5). The deepest area was surveyed in front of Section G and H at a value of about 12 m+PWD, reaching it's maximum in October 1997. This relates to a depth of 34 m below structure crest or to 26 m below the toe of the falling apron, respectively (Fig.4.5-6). These results support the estimated design scour depth of 12-14 m in front of the structure. The distance between the edge of the falling apron and the deepest point of the scour hole was about 50 m, resulting in a maximum slope in the order of 1V : 2H.

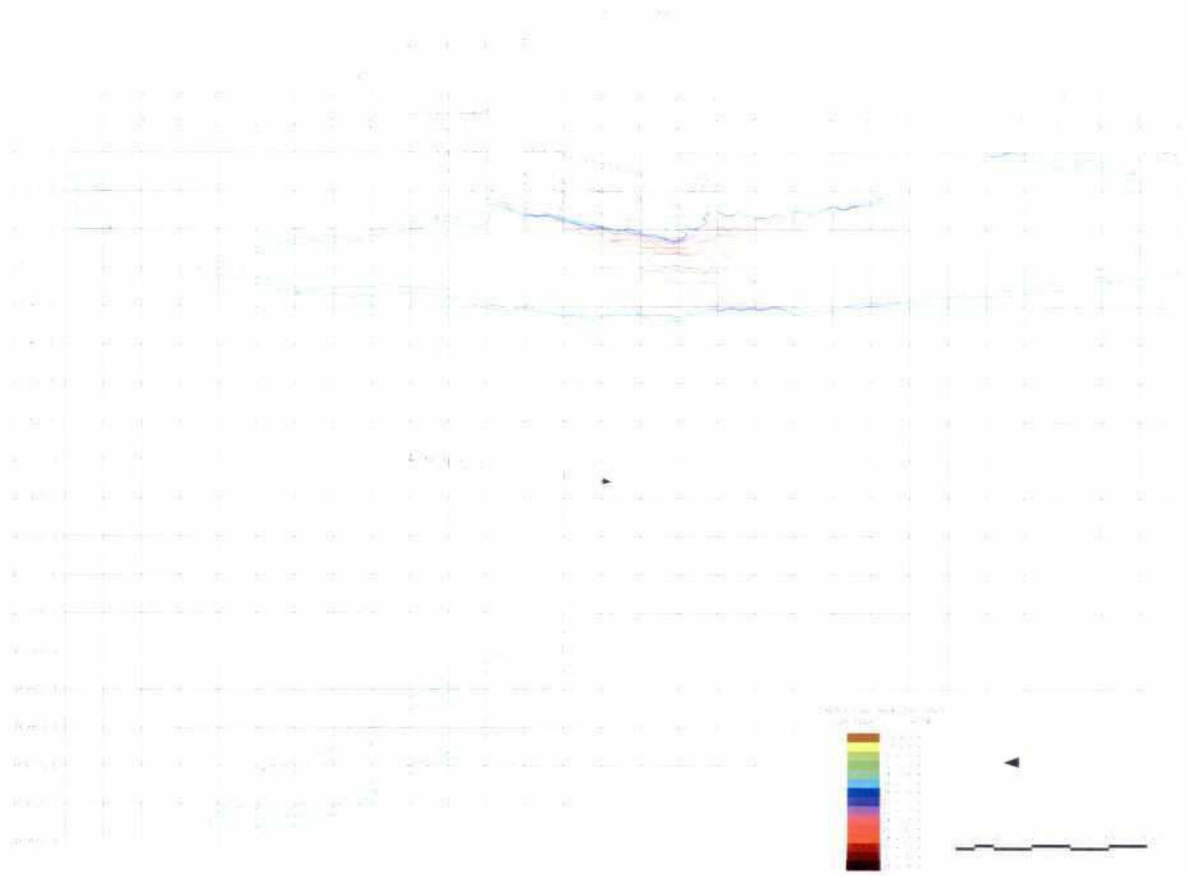

Fig. 4.5-5: Local scouring in the active flow channel in front of the revetment

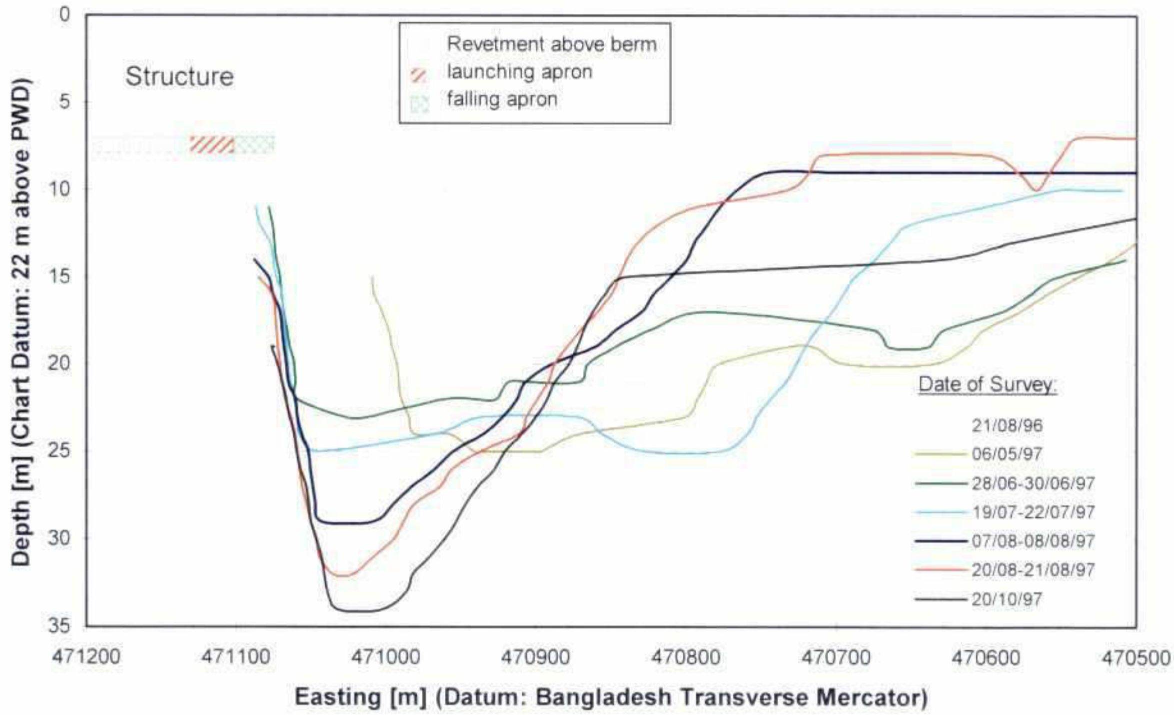

Fig. 4.5-6: Scour development in front of the test structure near to the downstream termination (monsoon 1997)

Due to the successive formation of a new main channel far away from the test structure (centreline about 1000m south west from Section H) the discharge in the flow channel at the structure face was reduced considerably which caused rapid accretion in this area (strong sedimentation between May and July 1998, compare Fig. 4.5-7), in some parts even accompanied by layers of sand accumulating above the launching aprons.

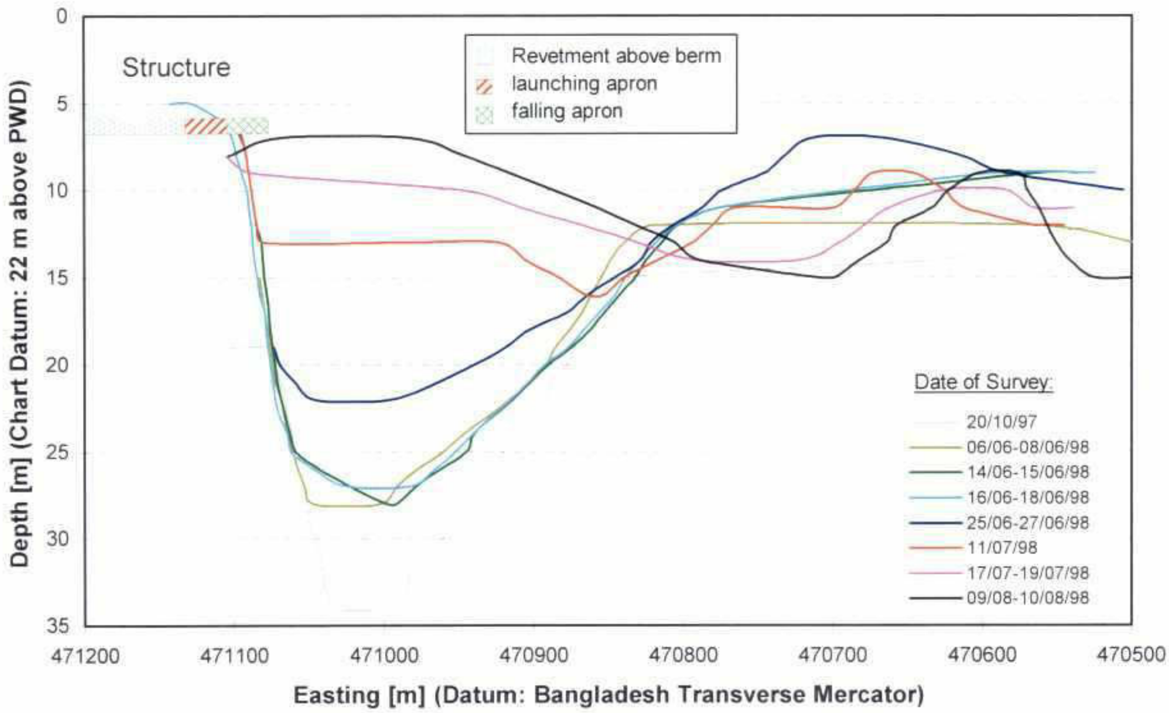

Fig. 4.5-7: Accretion of the scour bed in front of the test structure during the monsoon period 1998

4.5.4 Structure Stability

The intended development of the falling apron induced by flow attack during the monsoon period 1997 occurred almost over the whole structure length, whereas the partial development of the launching apron was concentrated mainly on test Section C (see Fig. 4.5-8). The scour in front of this section was relatively flat (Fig. 4.5-9). Nonetheless, it seems rather unlikely that this particular local situation was due to the particularly good performance of the geo-sand containers, or due to the large sand deposit functioning as a cofferdam during construction.

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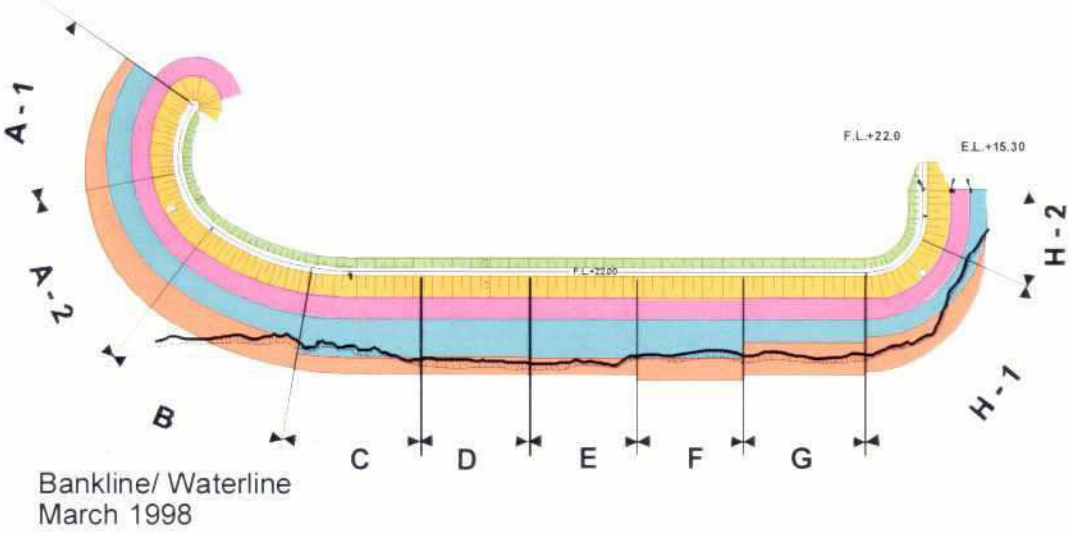

Fig. 4.5-8: Survey of the erosion progression along the structure (plan view)

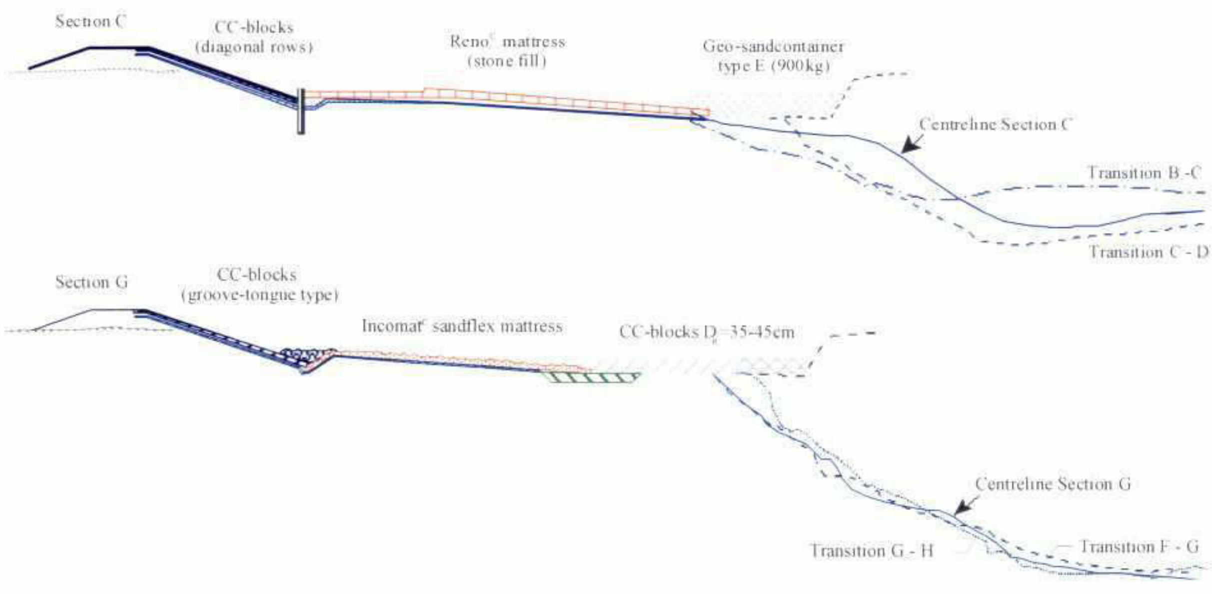

Fig. 4.5-9: Surveyed bathymetry at Section C (above) and G (below) of the revetment structure after the monsoon flood 1997

The Reno©-mattresses articulated as expected and retained full integrity, following the successive erosion of the flood plain. The launching part at the outer edge of Section C, shown in Photo 4.5-1, demonstrates the flexibility of such a system. It has to be emphasized that the success of launching apron systems depends mainly on the strength of the mattress units, the proper filling with ballast as well as the durable interconnection between the individual mattress units and the anchoring system. Based on the experience of the monitoring seasons till 1999/2000 this articulated mattress system may be considered as an easy to construct, economical and highly efficient system.

Photo 4.5-1: Partially launched Reno©-mattresses at section C after monsoon period 1997

When employing sand-filled geotextile mattress systems, the direction of overlapping of the individual geotextile sheets/ units is of importance. To avoid migration of soil particles from joints, all overlaps (of geotextile filter materials as well as of geotextile mattress systems) were arranged against the direction of flow. Nevertheless, sand loss occurred because the available filling material (Jamuna sand) is too fine to be fully retained by the geo-textiles. Thus, the overlapping parts were turned-over by shear forces. Two conclusions have to be drawn: (a) Gradual loss of fines from the filling material cannot be avoided. Therefore, the overlaps must be arranged in the direction of river flow. To avoid migration of soil underneath the geo-textile, the width of overlaps must be increased considerably. (b) Sand-filled mattress systems should not be utilised as permanent solutions since the gradual and continuous loss of Jamuna-sand fill may ultimately lead to a general weakness of the system. Improvement of the retention capability of geotextiles for mattresses is (at least at present) considered unfeasible by the manufacturers, not being justifiable from the cost-effective point of view. Delivery of suitably grained sand from the northern part of the country would inflate the cost and make such an alternative non-viable.

The falling aprons behaved as anticipated and protected the test structure. CC-blocks stabilised the scour slopes more effectively than geotextile sand containers, though the difference was marginal. The scour slopes, developed in front of the structure during the first season after construction, remained practically firm during the subsequent seasons despite rapidly changing situations caused by excessive scouring and depositing. This substantiates the suitability of falling aprons under the existing boundary conditions at the Jamuna river.

The protruding downstream part of the structure experienced the strongest flow attack and the main erosion in depth (Fig. 4.5-9, below), but taking into account the comparatively steep scour slope, the horizontal development of the scour hole was rather small. The good functioning of the CC-blocks prevented from more severe scouring so that the launching apron did not become exposed. Nevertheless, some repair was necessary in test Section H-1, where the falling apron was placed on geotextile filter materials which led to sliding of the protection material. Thus, parts of the area were exposed but without endangering the overall stability of the test section (Photo 4.5-2).

Photo 4.5-2: Exposed sub-layers after severe erosion during monsoon 1997 in front of Section H

4.5.5 Verification of Hydraulic Design Parameters

As already mentioned, the most critical hydraulic boundary conditions at Bahadurabad within the project period occurred in the monsoon phase 1997 due to the vicinity of an active main channel directly at the structure front. The recorded water levels referred to return periods of approx. 2 years only. However, due to the moderate variation in annual extreme hydraulic boundary conditions (water levels, flow velocities, etc.) at the test site, the theoretical values for return periods up to 25 years did not differ largely (compare with Kamarjani, where $u_2 = 2.9$ m/s and $u_{25} = 3.2$ m/s).

Maximum flow velocities right in front of the falling apron had been recorded at Section G (July 13, 1997, water level at 19.63m +PWD), showing values between 3.1 and 3.4 m/s at different positions over the water depth. This is reasonably below the assumed depth averaged design flow velocity of $\bar{u}_{des}$ = 3.5 to 3.85 m/s and allows for a certain margin of safety, covering local effects (e.g., eddy formation). Therefore, the general approach regarding the estimation of design flow velocities should be kept in mind in future design tasks.

The maximum recorded water level within the monitoring period (1994 to 2000) was 20.5 m+PWD. Overtopping of the embankment after completion was not observed but it has to be considered that the High Flood Level of 21.40 m+PWD (100-year return period) may possibly be followed by a considerable amount of wave overtopping, if severe wave action prevails at the same time.

Continuous recordings of wave heights at the Bahadurabad Test Site are not available. However, these are a prerequisite for a statistical analysis containing a comparison with the estimated significant design wave height of $H_{s,25} = 1.0$ m. The visual estimation of wave heights is rather difficult and subjective. Due to the very strong currents and substantial fluctuations in water depth over the fetch length, a reliable theoretical prediction is rather complex. Therefore, automatic wave measurements (using wave gauges) should be carried out at certain locations in future to obtain a better estimate of the existing wave climate. Nevertheless, the defined design wave height of $H_{s,25} = 1.0$ m is commonly used for the major rivers of Bangladesh and seems to provide a reasonable estimate.

4.6 OVERALL COSTS OF THE BAHADURABAD PILOT REVETMENT STRUCTURE

The overall financial investment costs for the Bahadurabad Test Revetment within the Pilot Project amounts to approx. 550 Mio Taka (about 11 Mio Euro), including land acquisition, labour, material, consultants work, etc. This does not reflect the cash flows required for future standardized revetment structures against bankline erosion because many cost components can be reduced considerably by implementing the lessons learned in terms of construction techniques and material used. Further details are discussed in Chapter 6 and 9, respectively.

4.7 GHUTAIL REVETMENTS

4.7.1 Structure Layout and Site Specific Aspects

The third test site was selected at Ghutail, located at the eastern bank of the Jamuna about 4 km south of Bahadurabad. The structure is close to 500 m and extends about parallel to the bankline. In general, the design procedures followed for the Ghutail Revetment Test Structure were similar to those at Bahadurabad. Nevertheless, in contrast to Bahadurabad, no critical superposition of different scour inducing processes was expected at this location. Therefore, the scour depth in front of the structure was estimated at a smaller design value of –5 m+PWD. The length of the launching apron was reduced significantly to approx. 16 m (Bahadurabad: close to 50 m), whereas the falling apron was designed rather conservative. This was done due to experience obtained during the inspections of the Bahadurabad Revetment, which led to the conclusion, that a sufficiently designed falling apron reduces the risk of continuous deepening of the river bed in front of the structure and thus decreases the demands placed on the launching apron. The upstream termination was not completed due to interference with the existing main embankment. Due to the relatively low risk expected in terms of the development of an unfavourable upstream embayment during the first years after construction, it was instead decided to extend the structure in downstream direction. In case of a critical development, the connection to the existing main embankment will be completed and sufficiently protected by BWDB authorities.

The layout of the test structure was kept principally uniform over it's developed length, only minor alterations regarding the size of the individual concrete blocks were employed in the falling apron design. The plan view of the Ghutail Revetment and the predominant set-up in cross section is depicted in Fig. 4.7-1 and Fig. 4.7-2, respectively.

As shown in the cross section, brick mattresses were used as a cover layer of the revetment slope (above berm), whereas RENO© mattresses and CC-blocks were implemented as launching and falling aprons, respectively. These modules were determined as the most appropriate solutions, resulting from the experience at Bahadurabad at the time of construction of the Ghutail Revetment. The construction techniques and the material used are described in Subsection 4.3.5.

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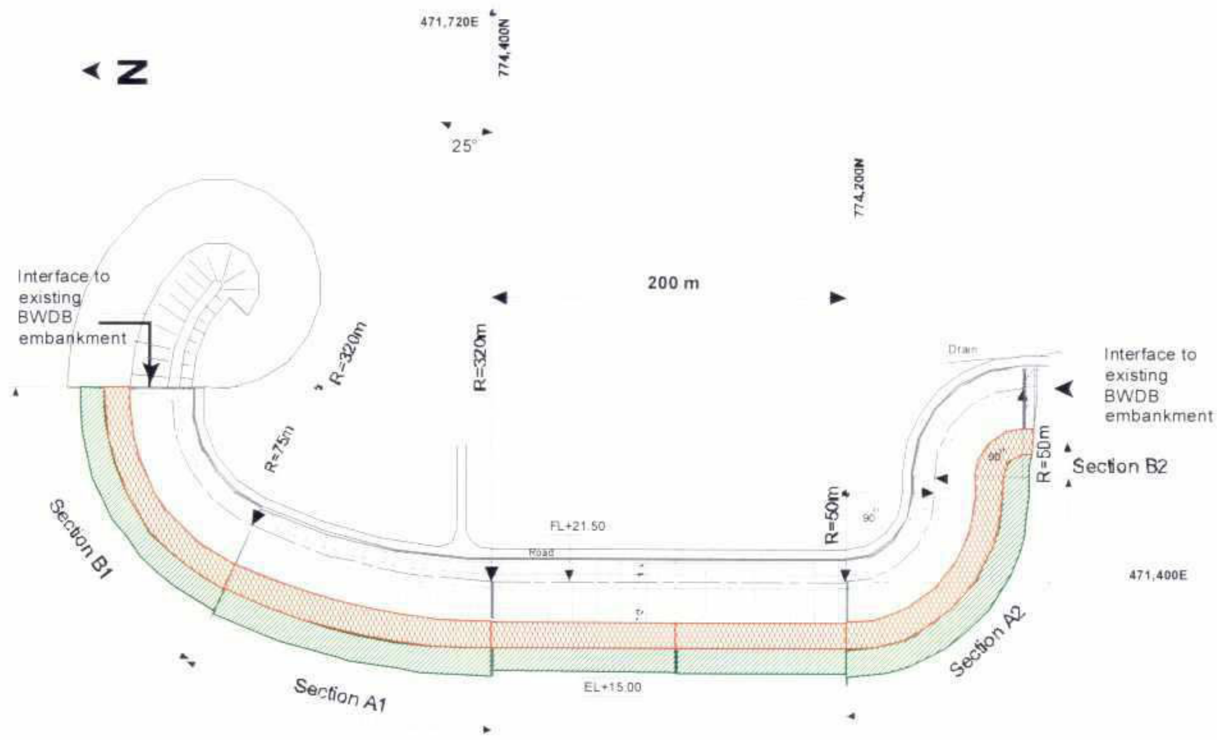

Fig. 4.7-1: Plan view of the Ghutail Revetment Test Structure

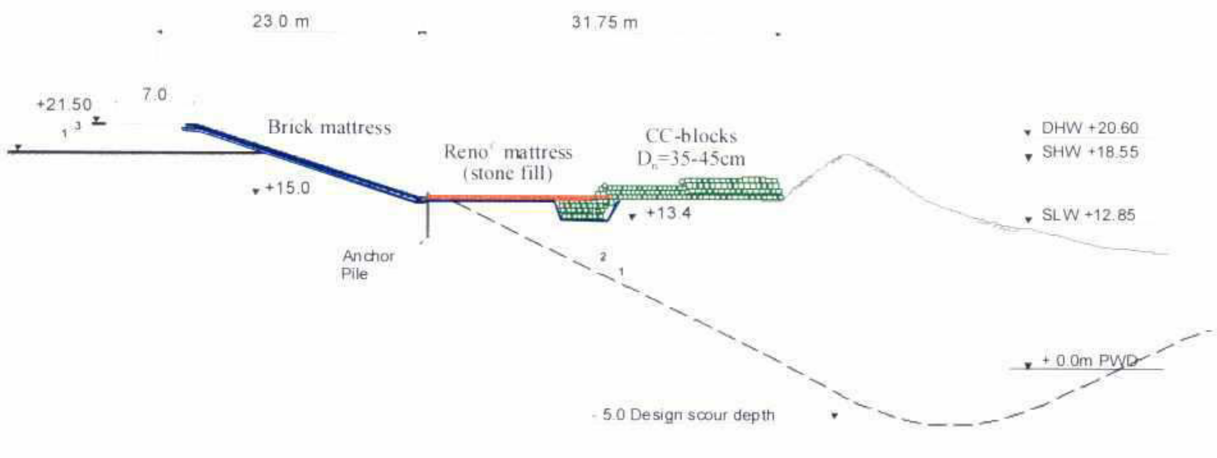

Fig. 4.7-2: Typical cross section, showing structure components at Ghutail

In Photo 4.7-1 a general view of the structure (camera pointing to north) is given, showing the randomly dumped CC-blocks (falling apron: I), the RENO© mattresses (launching apron: II) and the brick mattresses (revetment above berm: III).

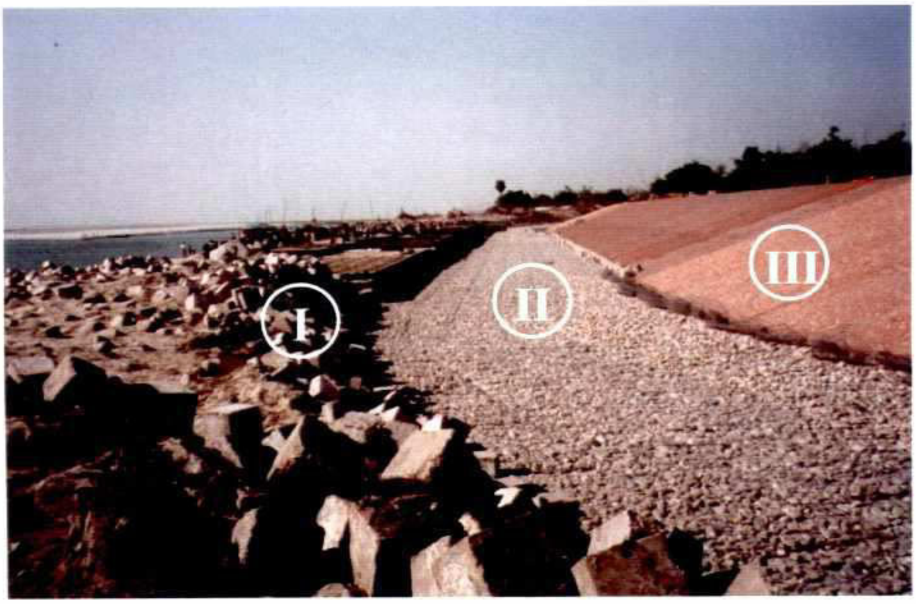

Photo 4.7-1: General view on the layout of the Ghutail Revetment Test Structure

After completion of the Ghutail Revetment, only the falling apron was subject to minor erosion. The maximum scour depth in front of the dumped CC-blocks was limited to 4 m+PWD. At present stage, no further conclusions regarding the structure stability can be drawn.

4.7.2 Overall Costs of the Ghutail Pilot Revetment Structure

Due to the experience gained from the revetment at Bahadurabad and the smaller dimensions of the structure, the overall costs at Ghutail were reduced considerably. Hence, the overall financial investment amounted to approx. 214 Mio Taka (about 4.2 Mio Euro).

5 SOCIO-ECONOMIC AND ENVIRONMENTAL EVALUATION

5.1 INTRODUCTION

The objective of the Project was to develop improved solutions for bank protection works against erosion, always with the aim to improve the living conditions of the local population. Therefore, the Consultant kept the well-being of the local population in the concerned areas in mind during all planning and construction activities. This is best documented by the fact that one of the decisive factors for the selection of suitable test sites was the "Something-to-defend" criterion.

Even during the Study Phase extensive socio-economic, agro-economic and ecological surveys as well as investigations were carried out in the pre-selected test site areas at Kamarjani and downstream from Bahadurabad Ghat. These investigations were intensified immediately after the final decision had been made to implement the proposed test structures. Special emphasis was placed on the investigation of

- social structures
- landholding distribution and farm fragmentation
- local institutions, public services, on-going social programmes
- psycho-sociological atmosphere
- agricultural activities

Further investigations included topics, an impact assessment and compensatory/mitigation measures like

- affected settlement areas and homesteads
- impact on agricultural activities
- land acquisition and compensation procedures
- resettlement issues
- agricultural and other socio-economic measures

Regarding the ecological assessment of the envisaged test sites, investigations were carried out during the Study Phase on

- the existing situation in terms of land use, flora, fauna, birds and fish, cropping patterns, water borne diseases
- possible risks resulting from the test structures on the environment

Follow-up surveys were carried out after completion of the first test structure at Kamarjani and its testing during the monsoon season 1995 as well as at the end of the Project again - at the first test site but then also at the Bahadurabad Test Structure at Kulkandi.

5.2 PRE-PROJECT SITUATION

The pre-project conditions in the selected test site areas were obtained through a Rapid Sociological Appraisal in 1992. The erosion rates in that year and the previous years were in the range of 150 to 220 m per year threatening not only homesteads and villages (like Dhutichara, Anelarchara, Bagharia on the right bank and Kulkandi, Belgacha, Ghutail Bazar on the left bank) but also cultivated fields and infrastructure.

Based on a population estimate in 1991 the average annual population growth since 1981 in the area of Kamarjani was about 1.1 % showing an overall outward-migration balance, whereas the corresponding figure downstream from Bahadurabad Ghat was about 6.8 %. This high increase of population density on the left bank was mainly due to the inward migration because of the on-going river bank erosion.

More than 60 % of the local population at Kamarjani and Ghutail Bazar were farmers and day labourers, in the area of Belgacha even more than 90 %. In the left bank areas, cropping system comprised rice, wheat, jute, mustard and additionally sugarcane, vegetables and spices. A downwards trend of livestock was noted in all investigated areas. The land value of the agro-ecological units was about Tk 148,100 per ha and the Net Value Added (NVA) about Tk 17,400 per ha with insignificant differences between the concerned areas.

Due to the permanent erosion during the past years the infra-structural development downstream from Bahadurabad Ghat was only slow and at Kamarjani even negligible.

As for the ecological investigations, during the Study Phase it was possible to use existing data on flora and fauna, although the database was rather incomplete. Therefore, field visits were undertaken for the collection of additional information, especially on fish, which resulted in a comprehensive inventory for the Jamuna river. The site investigations showed only minor ecological differences between the selected test site areas and it was finally concluded that negative impacts could nearly exclusively be expected during the construction phase.

5.3 PEOPLES PARTICIPATION

After identification of the test site areas in 1992, the Consultant started to involve the local population concerned. The aim of this participation process was to make the people of all social groups understand the proposed project, as well as to obtain information on the local social conditions, the value of land and on informal and on customary rights.

Participatory meetings were held during which main emphasis was placed on the following:
- Aim and objectives of the Project
- "test"-character of the structures
- local experience with the river and bank protection
- likely negative and positive impacts of the activities
- the attitude of the local people towards the Project and their willingness to co-operate
- issues of land acquisition and compensation payment

At all three test sites, it was difficult to make the people understand that the final decision on the exact location of the structures was mainly governed by technical and morphological considerations satisfying the requirements of test structures.

At **Kamarjani Test Site** it was possible to reach a consensus of opinion in favour of the Project before the start of the actual construction works, which was finally welcomed by the population. The villagers of Dhutichara and Anelarchara agreed in 1994 to provide their land and accepted the proposal of compensation money by the Project against one year term lease forgoing for the time being their legitimate demand of getting compensation money in full. That was the only way to start the work in time without the permanent acquisition of the land needed.

At **Bahadurabad Test Site** two alternative site locations were presented to a large cross-section of the local people in October 1995. One of them was finally accepted and consent of the people to provide their land on lease-basis was obtained as well. However, early November 1995 when the actual construction work had to start to make sure that the structure could be completed within the construction window of the dry season, the owners of the needed land did not make the latter available to the Consultant and his Subcontractor. As it is, site clearance was already delayed because a vested interest group hampered the work and tried to extract undue benefits from the Project. A large number of land owners were involved in the Project as subcontractors and put harmful pressure on the implementation of the work which finally failed, even after agreeing to pay house shifting costs, crop compensation and etc. The findings of the Study Phase that the latent structural conflict in this area between two socio-political clans stood on almost every issue proved to be well-founded. Work had to be stopped at the end of January 1996 and was postponed till the next dry season. The second attempt was made during the dry season 1996/97 and the structure was finally completed under very difficult social conditions.

The third **Test Structure** was built at **Ghutail Bazar.** This location was highly favoured by the Client and the Government of Bangladesh during the course of the selection process. Participatory meetings were held once again. Since, however, the structure had to be built within limited funds, it was rather difficult to make the people concerned agree to the selected location of the structure and the area to be protected. Even after the beneficiaries had accepted the structure, discussions and demands continued more or less till the end of the construction work, when the Government had finally decided to extend the revetment structure in the downstream area.

5.4 COMPENSATORY AND MITIGATION MEASURES

All project costs, except local taxes and cost for land acquisition, were borne by the Donors. However, as per provision of the Contract Agreement, the land needed for the test structures had to be provided by the Government of Bangladesh, i.e. by WARPO as the executing agency. Since the latter did not have any field unit to process the land acquisition, the responsibility was given to those BWDB field divisions under whose jurisdiction the relevant structure was located.

Possession of land for construction is a prerequisite for all project work. Since bank protection work is very much related to the bankline at the time of the actual construction, the process of land acquisition was very limited. A special document was needed to have access to the land of the project alignment before the actual acquisition was done and the land actually handed over to the Project by the relevant authorities. Therefore, site plans with an umbrella coverage of the area within which the actual alignment of the structures were expected to be set-up after recession of the flood were prepared. These site plans were transferred into relevant Mouza maps identifying precisely the affected land and the ownership. Moreover, inventories of affected households, shops, business units, tube wells, rural industries, trees, standing crops, fish ponds etc. were prepared. Immediately after fixing the exact alignment of the structures, the land plan and the inventories were modified from the umbrella coverage to the exact area. On the basis of the latter the land acquisition proposal was prepared in the required format and submitted to the Deputy Commissioner for formal acquisition of land.

With regard to the limited time available, the population was convinced in each case that the land should be made available to the Project before the actual payment of compensation by the authorities. It was finally agreed that the Project had to bear the cost for house shifting and compensation for trees and standing crops as per agreed rates. On receipt of the compensation, the owners had to vacate the land latest by beginning of November so that the actual construction work could begin accordingly.

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The programme worked well at the Kamarjani and Ghutail test sites but not at Bahadurabad. The construction work at Bahadurabad were delayed due to non-availability of land from the owners in the dry season 1995/96. Therefore, the construction works had to be suspended in early 1996 and postponed till the next dry season. This, however, required a second phase of land acquisition for the same structure because the location of the revetment had to be adjusted to the bankline development during the monsoon season 1996.

The total areas to be acquired for the test structures were as follows:

- Kamarjani: 13.84 ha
- Bahadurabad (Kulkandi): 14.65 ha (first phase in 1995/96)
  15.98 ha (second phase in 1996/97)
- Ghutail: 6.21 ha

The compensation for house shifting, standing crops, trees and fishery were at

- Kamarjani: Tk 2.51 million
- Bahadurabad (Kulkandi): Tk 3.70 million
- Ghutail: Tk 0.30 million

In addition, costs for reconstruction or shifting of public institutions were supported by the Project as below:

- primary school at Kamarjani: Tk 50 thousand
- high school at Kulkandi: Tk 1.8 million
- primary school at Kulkandi: Tk 1.5 million
- Madrasha school: Tk 1.1 million
- hospital at Kulkandi: Tk 1.5 million
- construction of orphanage at Ghutail: Tk 90 thousand

5.5 POST-PROJECT SITUATION

A sample survey of the people living in the relevant project areas was undertaken in March 2000 to assess and evaluate the post-project conditions. The land value per ha has increased considerably in all the test site areas because the bank erosion was stopped by the structures.

At **Bahadurabad Test Site** (Kulkandi) the average value per ha was Tk 207,500 and at **Ghutail** even Tk 506,000. This is an increase of about 140 % and 340 % respectively. The average Net Value Added (NVA) was found to be Tk 45,000 at **Kamarjani** and Tk 52,000 at **Bahadurabad** (Kulkandi) and **Ghutail Bazar** which is an increase of about 300 %.

Moreover, it is worth mentioning that the livestock in the three areas concerned is stable now and that a rapid infrastructural development could be observed. Hence, it can be concluded that the bank protection structures brought significant improvement in terms of hydro-morphological development, infrastructural development, land value and Net Added Value of the agricultural product.

5.6 ENVIRONMENTAL ASSESSMENT

An ecological assessment was made during the Planning Study Phase in the pre-selected test site areas. The investigations included the existing environment at that time and an analysis of possible impacts of bank protection structures and other river training works on the environment. Advice was given regarding minimising negative impacts which were more or less expected solely during the construction phase but not after completion of the structures.

Follow-up surveys were recommended to be carried out after completion of the structures, the observations and conclusions of which should provide basic data for the impact analysis of the structures.

A first survey of the biological and the human environment was undertaken at **Kamarjani** in November 1995, just after the Groyne Test Structure had managed to withstand the first monsoon season during which it was endangered by the river.

Both test site areas were investigated in more detail during the last quarter of 1999 in order to assess the environment response to the structures 4.5 years and 2.5 years respectively after their completion. For further details, especially regarding the investigations of 1999, refer to Annex 3.

During the 1995 survey at **Kamarjani Test Site,** the conclusion was made that the erosion of the riverbank, the flood plain and the hinterland would have been considerable without the groyne field and land, fields and homesteads would have been washed away, leading to a disaster for the local population. Although some cultivated land had to be used for construction, people benefited substantially from the structure which not only stopped the erosion process in that area and protected the cultivation behind the embankment, but also provided shelter for people, cattle and houses during the high flood period.

For the assessment of the biological environment the terrestrial vegetation, aquatic plants, fauna and fish were investigated. Some trees and bushes had to be cut at the construction site and no terrestrial vegetation had yet started to grow around the borrow pits behind the embankment, because the fertile soil had been removed. Nonetheless, some borrow pit owners had planted Dholkalmi around the borrow pits. This plant grows quickly and provides shelter to the fish. Weeds and seedlings of shrubs and trees were found to grow on the slopes of the embankment. The highest development of plants was observed in the lower part of the brick mattressing, where the deposition of silt was larger than in other areas. Some plants were even growing between the boulders and cc-blocks. Durba grass on the slope of the embankment had developed normally but was damaged by wave action and current.

The development of aquatic plants in the borrow pits and near the riverbank was limited due to the time factor since the survey was carried out only a few months after the construction activities. Since the completion of the structure time period was too short to carry out a reliable impact assessment the fauna. The same can, more or less, also be stated for fish. However, the transversal migration of fish was delayed and hampered by the existing embankment.

During the investigations of 1999 the environmental issues and related impacts were considered within three categories viz. physical, biological and human. Regarding the physical environment it was confirmed that the structure has protected the cultivated land behind the embankment since its completion, whereas downstream erosion has continued. The extent of char formation, since 1995, has not varied and no permanent loss or gain of land in the Project area was observed. Fields are protected by the structure from early flooding and the soil humidity can be maintained for a longer period. Thus, the capability and fertility of the land has considerably increased and can be improved to a larger degree by irrigation from the borrow pits. No negative impact on the soil characteristics was noted. This can also be stated for the ground water level and quality because of the relatively small scale of

the structure compared with the dimensions of the river. From this point of view a significant impact on the river water quality may also be excluded.
Regarding the flora, widespread planting of Dholkalmi has been noted. It seemed that the plant had propagated itself naturally alone on some stretches of the embankment. Though the structure provides some scope to plant trees, no attempt has been made for afforestation behind the embankment.

No significant impact on bird and fish communities could be noted. Due to the increase of the population, terrestrial fauna decreased to a low level, except rats and other rodents for which the embankment provides an excellent habitat. However, the stability of the embankment is endangered by their activities.

Since the groyne structure provides protection for housing, fields etc., there is definitely a positive impact on the human environment. Not only the landowners benefit from the Project but also others due to improved employment and income. Crop cultivation, rice production and fishing, especially in the borrow pits, have been enhanced which led to an improvement of nutrition and the general state of health of the local population.

The impact on the physical environment of the second test site at **Bahadurabad** (Kulkandi) is very positive since the erosion of the riverbank was stopped by the Revetment Test Structure and the village behind is well protected, whereas erosion upstream and downstream continued during the years after completion of the structure. Cultivated land and homesteads are safe and no loss of land has been observed. Since the land is normally not flooded by the river during the monsoon season, the structure does not have any influence on the fertility of the soil and its humidity. The structure has not influenced neither the level nor the quality of the ground water.

A positive impact on flora and fauna was observed in 1999. More trees were planted in the homesteads and behind the embankment which is, among others, a benefit for bird life. Since the land is protected from flooding, fish cultivation in ponds is safe and has increased. No negative impact on fishing in the river was noted.

Also the impact on the human environment is positive. The village Kulkandi (Bahadurabad Test Site) is fully protected against erosion by the structure and hence, the landowners and other people have received a positive impact, because of improved employment and higher income. This is best proved by the increasing tendency of immigration of people to Kulkandi.

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6 ECONOMIC EVALUATION

6.1 MAIN OBJECTIVES AND METHODOLOGY

The financial evaluation will cover two main aspects: Firstly, to provide a presentation of the project costs (i.e., to point out the use of resources) and, secondly, to provide a sound basis for a cost-benefit-analysis (CBA). The protection measures considered in this pilot project were designed as test structures consisting of several sections with different structure components, to study optimal properties in terms of function, stability and implementation. Furthermore, a substantial effort was put on research, including model testing, reporting and meeting between experts. Nevertheless, due to the innovative nature of the project, some setbacks occurred within project implementation caused by various reasons. Therefore, the costs within the pilot project may differ considerably when compared to a conventional planning and implementation process or, even more significantly, when compared to an optimized design with a cross-section almost identical along the structure length. Hence, it is necessary to evaluate an optimized structure type (function and costs) to provide a basis for future investments. Consequently three subsequent cost frames have to be considered:

i) **total investment costs pilot structures:** evaluation of costs based on experience with the pilot project (i.e. all actual cash flows)
ii) **adjusted investment costs pilot structures:** evaluation on basis of corrected values, eliminating avoidable setbacks ("lessons learned") and costs not directly related to the structure implementation (research, etc.)
iii) **investment costs for standardized protection structures:** evaluation of costs based on future standardized structures (restricted materials, optimized solutions and construction methods)

The CBA of (iii) is given in Section 9.6 (standardized protection structures). The methodology used for the project evaluation of FAP 21 follows in general the standards given by FPCO Guidelines for Project Assessment (1992) including the updated Annex 1 (1994). Nevertheless, some modifications in the approach were necessary to consider specific aspects of the bank protection pilot project.

6.2 GENERAL ASSUMPTIONS AND BASES FOR CALCULATIONS

6.2.1 Discount Rate and Period of Analysis

The standard discount rate to be considered for future investments and profits was fixed at 12%, as defined by FPCO, to make permit the comparison of the various projects investigated within the FAP programme. The period of analysis chosen for the economic evaluation of the projects was set to 30 years, beginning at the start of the implementation (year 0) as recommended by the FPCO guidelines. Despite the fact that the fiscal year is defined as the 12 month period starting on July 01 of each year (BWDB), the costs within the project have been related to the Christian calendar. For reasons of simplicity, it may be assumed that all cash flows (costs and benefits) take place at the end of each year included in the period of analysis.

6.2.2 Reference Year

In order to adjust the monetary costs and benefits occurring at different phases within the evaluation period, all current prices are given as constant prices in year 2000, independent of the actual completion year of the structure, applying the Bangladesh general index of prices for building construction (see Table 6.2-1, adopted from Statistical Pocketbook, Bangladesh, 1999). Future price escalations beyond year 2000 cannot be taken into account because their prediction would have to remain rather vague.

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Year	1993	1994	1995	1996	1997	1998	1999	2000
General index	1540	1547	1657	1763	1810	1897	1999	2101 *)
Inflation factor	1.36	1.36	1.27	1.19	1.16	1.11	1.05	1.00

Source: Statistical Pocketbook, Bangladesh 1999 *) estimated

Table 6.2-1: Basic indices for price escalation in Bangladesh (1993-2000)

6.2.3 Prices and Conversion Factors

When available, standardized prices based on rates given in the FPCO guidelines or other official publications (e.g. the statistical year book of Bangladesh) were used for the calculations. Other data, like harvest yield where also taken from these sources. When necessary, the average standard values were adapted to more realistic values taking into account the local situations by surveys and interviews with stakeholders, undertaken within the FAP 21 project.

The transformation from financial to economical prices was done by applying weighted average conversion factors (ACF) on the costs of the test projects, following the definitions for specific conversion factors by FPCO. Due to the different equipment and material used, the weighted average conversion factor varied between Kamarjani ($ACF_K = 0.77$), Bahadurabad ($ACF_B = 0.74$) and Ghutail ($ACF_G = 0.71$) test sites. Considering the fact, that the estimation of future benefits bears diverse uncertainties and conversion factors are subject to change, it was decided to approximate the economic value of potential benefits directly.

6.2.4 Identification of Costs and Benefits

For the cost-benefit-analyses (CBA) two scenarios were studied: Firstly, the case without any erosion prevention ("without case") and secondly, the development of the designated area, when efficient measures against bankline erosion were taken ("with case"). In this context it has to be emphasized, that the structures within FAP 21 where not designed to prevent the flooding of agricultural or rural areas but, rather, to prevent the loss of land and infrastructure due to erosion induced by strong currents occurring at high and low water levels at the banks of the Jamuna river.

The financial analysis of the total investment costs for the pilot structures includes all costs incorporated in the implementation of the Test Structures at Kamarjani, Bahadurabad and Ghutail. Substantial elements to be considered are costs related to land acquisition at the respective locations, to the design and construction of the test structures (including all personnel and equipment costs), and to supplementary costs associated with monitoring, maintenance and repair. Further details will be described within Subsection 6.2.5.

The adjusted investment costs for the pilot structures will consider an identical layout as compared to the first case, but are reduced by extremely high extra costs related to research, unfeasible construction techniques and damages by early floods, etc. The cost adjustments are site-specific and, therefore, described in Section 6.3.5, Section 6.4.5 and Section 6.5.5, respectively.

Due to the fact that the structures are located particularly in rural areas, the estimation of the benefits gained by such measures is particularly difficult and the result is often of poor economical value. A large part of the profit is related to intangible assets and to further development of the area scrutinized. From this point of view and - considering the dense population of Bangladesh - the investment is a governmental responsibility which follows other goals that differ from private investment. However, it is essential to give a sound analysis in order to allow for the weighing of costs between alternative structure types and between the importance of different locations considered for implementation.

6.2.5 Assessment of Costs (Actual Cash Flow of Pilot Structures)

6.2.5.1 General

In the cost assessment, all cash flows which occurred within the planning and implementation phase have been considered. The total sum is split up into costs related to land acquisition (borne by GoB), to expatriate and local consultants, to actual construction work (including labour, material and equipment) as well as for the adaptation measures which were necessary to enhance the sustainability of the structure. Moreover, the monitoring and maintenance costs are specified.

6.2.5.2 Land Acquisition/Compensation

The plots of land required for the implementation (base) of the test structures and for the site facilities were owned by local people, therefore land acquisition was necessary. Despite the fact that the costs for land acquisition and compensation were borne by the government, they have to be considered in the evaluation. The total amount of these costs can be split up in permanent sale of property (inevitable, because occupied by the structure itself) and temporary lease (used for temporary site facilities), paid as compensation for loss of potential earnings on crop land (crop failure). In addition, the costs for house shifting and for the numerous fishponds were paid to the owners. The land costs differ quite significantly among the site locations, depending mainly on the attainable benefits (e.g., number of homesteads, level of infrastructure, type and intensity of land use, cropping pattern). The governmental rates for compensation were calculated on the basis of average prices plus an extra charge of 50%, considering the fact that it was not the primary intention of the land owners to sell their property. This step was necessary to obtain the support of the stakeholders and to allow for a quicker realisation of the commencement of the project.

6.2.5.3 Consultants

The fees and allowances of expatriate and local consulting work were split up between the three test sites, taking into account the percentage of each pilot structure on the total overall project construction costs. In the total amount, costs for hydraulic and numerical model testing as well as for advisory services (etc.) are included.

6.2.5.4 Construction Costs

For determining the economic efficiency of the test structures, the expenditures for the execution of construction work (e.g. procurement and supply of steel pipes, construction steel, concrete elements, stones, boulders and geotextiles, hiring of equipment, etc.) and for personnel (expatriate and local consultants as well as for local supporting staff) were taken into account. Other costs, such as port charges, demurrage and contingencies, were also included.

Annual cost flow of construction costs for the test structures at Kamarjani, Bahadurabad and Ghutail are computed from the actual cost compilations provided in the Administrative and Financial Reports, referring to Progress Reports No. 19, 20, 26 and 29 of the FAP 21 project, respectively. In this context, it should be mentioned that the allocation of costs given in this assessment follow a more technical rather than accountant's interpretation, therefore minor differences may be apparent.

6.2.5.5 Adaptation Costs

Adaptation measures were initiated either as preventive action or, in case of partial failure, for

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structure components. For the cost assessment, the adaptation measures and costs were defined as those resulting from significant modifications and adjustments in the original design of the pilot test structures to cater for existing hydraulic loads in order to sustain their stability and function. On the contrary, whenever the original design remained identical afterwards, the costs were taken as "maintenance and repair" (refer to Subsection 6.2.5.7). In contrast, the Groyne Field in Gaibandha district was extended in length (1995-96) to protect the newly built Manos regulator. However, the costs for the modification were taken as construction costs because they were not induced by damages of any structure component.

6.2.5.6 Inspection and Monitoring Costs

Inspection costs were related to checks of the structure itself (damages to protective layers, main embankments, etc.) which were important to arrange for immediate rectification, if necessary. The purpose of the monitoring programme was to scrutinize the meteorological and hydraulic boundary conditions and the morphologic changes in the vicinity of the structure locations. The latter was a crucial issue to prove assumptions made while designing the test structures and to find out about the actual limitations in terms of structure stability. Due to the pilot character of the FAP 21 Project, the inspection and monitoring costs accrued over time are relatively high and, therefore, do not represent an appropriate order-of-magnitude for regular bank erosion protection.

The total expenditures for monitoring within the FAP 21 programme have been suitably allocated to the Kamarjani and Bahadurabad Test Sites (Ghutail started later). Differences in the total amount used for monitoring between both sites are caused by site specific requirements, duration of monitoring activities etc.

6.2.5.7 Maintenance and Repair

Maintenance and repair is obligatory to ensure structure safety and to allow for a sustainable use of budgetary resources. The wear and tear during the life of structure elements requires the prediction of impact and damage following a probability function. Due to the fact that the Jamuna river is highly mobile and changes it's course frequently but is unpredictable up to the present, this procedure is not appropriate for the quantification of maintenance and repair costs. Besides, natural impacts and human intervention (i.e. the unauthorized removal of boulders or concrete blocks) may contribute to this cost segment. Therefore, as an alternative, the future maintenance costs have been approximated by a percentage (4%) of the expenditure of the cover layer and toe protection which is quite common in civil engineering projects.

6.2.6 Assessment of Benefits

6.2.6.1 Agricultural Production

Due to the reduced risk of loosing property by means of bank erosion, it can be assumed that the agricultural production will increase. This process needs further investment and, hence, time to develop. At this stage, predictions concerning the development of the specific areas can only be imprecise. Therefore the average growth rate recommended by the FPCO guidelines (3 – 4 %) has been adopted. In order to prevent double counting, this positive effect is only considered in the calculation of prevented damage (Subsection 6.2.7).

6.2.6.2 Fisheries

A similar approach can be attributed to the future development of fishing opportunities (artificial fish ponds). It is uncertain, whether the total area of fish ponds or the catch rate will actually increase and whether this effect can be directly related to the bank protection measures. Furthermore, only the infrastructure (i.e. the ponds and irrigation canals) is protected, whereas the catch will presumably be lost when flooding occurs. On the other hand, a relatively large compensation amount was paid for loss of fishing grounds. This has been taken into account as potential lost property, which is in agreement with FPCO guidelines for project evaluation.

6.2.6.3 Others

Other positive impacts on the infrastructure (e.g. the road at Ghutail), or on the social development (schools, mosque, orphanage) which were financially supported by the project, are not considered in the assessment of benefits but certainly had positive effects on the overall development of the region.

6.2.7 Reduction/Prevention of Potential Losses

6.2.7.1 Casualties and Intangibles

People living in the flood prone areas of Bangladesh are used to escape the threats of flooding - most homesteads are equipped with a boat, which can serve as a lifeboat. Despite this, casualties occur during the major floods in Bangladesh and it is anticipated, that bank protection measures could contribute to reducing this number considerably. This factor has to be taken into account by the decision makers but will not be taken into account in the loss analysis.

6.2.7.2 Loss of Property

Considering the fact that the Test Sites are located in more or less rural areas, no large multi-story buildings or industrial complexes have to be accounted for. For this reason, the potential loss of property includes primarily

- loss of land;
- damage/destruction of houses and dwellings;
- damage/destruction of public, private and agricultural infrastructure;
- loss of trees, crops and harvest yields;
- dislocation costs, and other assets.

Only at the village of Ghutail, which serves as a regional trade centre, a larger number of concrete/ brick buildings as well as shopping areas is existent.

Since no detailed data on the above listed total or partial damages and on the associated losses related to erosion are available (it would be necessary to distinguish between flood and erosion damage), the prices, estimated on the basis of post project surveys are employed in the analysis. Despite the fact that many discussions and interpretations regarding the inclusion of property value exist amongst economists, this decision was made because the compensation costs are strongly influenced by property speculation prior to the land acquisition and are dependent on the area used for the structure implementation which was particularly highlighted during the land acquisition process at Bahadurabad Test Site (compare Subsection 6.4.1.1). The average local land price determined for each specific test area is supposed to include all above listed aspects, hence, it may be considered a relatively vague parameter. On the other hand, it is hardly predictable where the property value will accumulate after successive erosion of the bank line ("with out" case). Within the context of bank erosion prevention projects, it is justified to assume that the actual embankment line will always be populated most densely, thus the approach taken in this report is rather conservative and based on data previously gained. Possible misinterpretations in terms of property assessment will be scrutinized within a specific sensitivity analysis.

6.2.7.3 Damage/Destruction of Existing Main Flood Embankments

The traditional practice to cope with destroyed main embankments due to river erosion is to retreat the defence line further inland. It is assumed that an average distance of 1000 m between the formerly existing and the newly erected main flood embankment is an appropriate value. Depending on the

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erosion rate of the specific location this potential loss will be introduced as a benefit every n^{th} year (with n = 1000 m/ (erosion rate/year), considered as an integer).

6.2.8 Bankline Recession

As already mentioned, a probability function (e.g. a non exceedance level of flooding) cannot be adopted for the cyclic macro-temporal and spatial distribution of erosion along the bank line of the Jamuna river. Instead, for a more integral approach, the recently measured erosion rates and the predicted future bank line development at the different locations, taken from Chapters 3 and 4 will be utilized in the calculations. Due to the considerable uncertainty in future erosion rates this parameter has to be investigated in a sensitivity analysis when determining the economic internal rate of return (EIRR) of the Project.

6.2.9 Potential Disadvantages Induced by Protective Structures

Potential disadvantages within the immediate surroundings of the protected area seem to be are not worth mentioning. Flooding may occur, independently from the measures taken within this project, so that the cropping areas will get fertilized by river sediments as before project implementation. Negative effects might occur in neighbouring areas due to change of the river course and lee erosion, however, these must not necessarily be attributed to the structure only.

6.2.10 Data Sources

Data used as basis for the calculations are adopted from the following sources:

- FPCO guidelines: standardized rates and conversion factors
- FAP21 Administrative and Financial Reports: actual investment costs of the pilot projects
- Statistical Pocketbook Bangladesh 1999: general index, agricultural data
- Experience from similar projects for cross checking of assumptions
- Governmental data (BWDB), interviews with stakeholders: land prices

6.3 ECONOMIC EVALUATION OF THE GROYNE FIELD AT KAMARJANI TEST SITE

6.3.1 Preliminary Remarks

The design of the Groyne Test Structure at Kamarjani site under Gaibandha district was carried out between September 1993 and February 1994. Construction started at the beginning of October 1994 and was substantially completed during the month of April 1995. The structure was monitored during five consecutive floods amongst which the first flood in 1995 caused damages. Hence, the structure had to be adapted subsequently by appropriate corrective measures. The total length of the groyne field comprising a series of seven groynes in is about 1700 m including connecting revetments (see Fig. 3.3-1).

The area of interest of this project is situated in the Sadar Upazila of Gaibandha district, Ghagoa and Gidari Unions and some parts of Kamarjani Union. A sample survey of the people living in the area was undertaken in the month of March 2000. The results show that most of the people here are small and medium farmers, nearly half of the household heads have education below primary level, around 70% have agriculture as major occupation, the rest being in business, fishing and other trades. Major crops cultivated are Irri/Boro, Aman, wheat and oilseeds, having less than average yield per/ha. No remarkable change in the cropping pattern and yield rate before and after the project was observed in the project area. The primary benefit is the value of land that would have been lost (without scenario). The surveyed people reported that the value of agricultural land increased about three to four times after the completion of the project. Other secondary benefits which may be attributed to the groyne structures are the infrastructural development and the improvement in the income level of the people in the project area. Semi-pucca and tin-shed houses were built, in some cases development and repair of schools and shops took place. Although it is difficult to estimate the economic returns of these indirect benefits of the structure, nonetheless they are quite obvious and visible in the entire locality. Furthermore, the newly built Manos regulator, previously washed away by the Jamuna, was aimed to be protected by the groyne field.

6.3.2 Assessment of Project Costs

The total capital investment employed in the implementation of the Test Groyne Field is shown in Table 6.3-1.

Item	Taka	Taka
Land acquisition/ compensation		
Land (groyne area)	853,556	
Land (embankment)	2,395,106	
Houses/ ponds/ trees/ crops	[1]	
Temporary land lease, etc.	2,500,000	
Administration costs	324,866	
Sub-Total (land acquisition/ comp.)		**6,073,529**
Consultancy		
Expatriate staff	95,642,942	
Local staff	19,981,882	
Sub-Total (consultancy)		**115,624,824**
Construction (1994 - 1995)		
Embankment (revetm. layer)	17,526,199	
Embankment (earthw., BWDB)	10,341,592	
Groynes	88,139,433	
Revetment and scour protection	46,568,451	
Equipment procurem. and hire	60,179,554	
General [3]	64,875,200	
Sub-Total (construction)		**287,630,429**
Adaptation/Extension (1995 - 1997)		
Groynes	23,646,386	
Revetment and scour protection	32,622,102	
equipment procurem. and hire	3,440,994	
General [3]	58,530,076	
Sub-Total (adaptation)	**118,239,558**	
Extension (groyne G-A/2)	16,839,836	
Sub-Total (adaptation/ extension)		**135,079,394**
Monitoring/ Maintenance (1995 - 1999)		
Monitoring	24,071,030	
Maintenance/ repair	37,437,038	
Sub-Total (monitoring/ mainten.)		**61,508,068**
Grand Total (Investment costs)		**605,916,244**

[1] not detailed, included in land prices [2] resettlement costs, etc.

[3] costs for site facilities, camp, port charges, insurance, etc.

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Table 6.3-1: Total capital investment for the Kamarjani Groyne Field (financial current prices)

The costs are given in financial current prices. Additional information on the temporal occurrence of cash flows within the implementation period is given in Attachment 1. The site specific cost components are given in detail in the following subsections.

6.3.2.1 Land Acquisition

The land required for the construction of the groyne test field was financed by GoB (through FPCO/ WARPO). An amount of Tk 2,500,000 was required as compensation money for temporary loss of crops/ harvest yields and shifting of houses in areas used for site facilities. The rates applied are weighted average rates for the mouzas Analerchara and Dhutichora. A distinction is made between costs for the embankment area and for the groyne area to allow for the evaluation of other cost scenarios (see Section 6.1). The costs for land acquisition at Kamarjani Test Site are summarized in Table 6.3-2.

Item	Quantity [1]		Rate (Tk)[2]	Amount (Tk)
Groynes				
Homestead	4.58	acre	101,366.00	464,256
High Land	2.19	acre	47,845.00	104,781
Low Land	-		-	-
Roads	-		-	-
Sub-Total (a)	6.77	acre		569,037
Comp. market rate	-		50% of sub-total (a)	284,519
Sub-Total (A)				**853,556**
Houses	-		-	-
Other structures	-		-	-
Ponds	-		-	-
Trees	-		-	-
Crops	-		-	-
Sub-Total (B)	-		-	-
Sub-Total (A-B)				**853,556**
Add. Admin. Cost	-		10% of sub-tot. (A-B)	85,356
Total (groynes)				**938,912**
Embankment and borrow pits				
Total (embankment)	27.54	acre	(lump sum)	**2,634,617**
Total (land acqisit.)				**3,573,529**
Compensation costs				
land lease, etc.[3]	-		(lump sum)	2,500,000
Grand Total	**34.31**			**6,073,529**

[1] 1 acre = 0.405 ha

[2] Total quantities/ weighted average rates for mouzas Analerchera and Dhutichora

[3] For temporary use during implementation, incl. house shifting/ loss of crops, etc

Table 6.3-2: Land acquisition costs at Kamarjani Test Site (area for 7 groynes and embankment)

6.3.2.2 Construction Costs

The construction of the initial groyne field at Kamarjani was carried out in the dry season 1994/1995, by local sub-contractors with support from expatriate advisors for special heavy equipment. The equipment and the piles for the permeable groynes cover another large part of the expenses. Additional costs which originated from earthworks for the main embankment were borne by GoB.

6.3.2.3 Adaptation Costs

Due to the rather vague definition it is sometimes difficult to make a clear distinction between costs associated with maintenance and costs related to adaptation. In this context, the replacement of impermeable parts of the groyne by permeable sets of tubular piles were taken as adaptation costs. The extension in length of the groyne field (in bankline parallel direction), created by an extra groyne (groyne G-A/2), was carried out to protect the Manos regulator, therefore, the related costs are given separately (see Table 6.3-1).

6.3.2.4 Monitoring Costs

The major part of monitoring was not directed to the pilot structure itself but to the measuring of the hydraulic boundary conditions and the bathymetry in the near field of the Groynes and in the neighbouring river channels. The equipment was used for Kamarjani and Bahadurabad, therefore, costs of monitoring and inspection were mostly shared between both Test Sites at a rate of 50% each. Future monitoring has been calculated at a rate of Tk 1,820,000 per year (Annex 6).

6.3.2.5 Maintenance and Repair

Maintenance and repair works were necessary within the first years after construction. A substantial amount was used for reinforcement of groynes and for supplementing the toe protection against scouring. Included in the sum is a considerable portion (approx. 7.9 million Tk) of surplus protection material (concrete blocks and boulders) not used till now, but stored at site, serving as a reserve for future emergency measures. Future annual maintenance costs are estimated at a rate of Tk 2,563,800 (4% of cost of cover layer and toe protection).

6.3.3 Assessment of Benefits

6.3.3.1 Prevention of Potential Losses

Instead of applying a probability function of structure failure, the estimated annual erosion rates will be used to describe the prevented losses in the "with"- scenario. Table 6.3-3 summarizes the average erosion rates for different time periods, taken from bankline surveys between 1988 and 1999. As a result, an average weighted annual erosion rate of e = 191m per year was used for the following calculations.

Period of time[1]	Number of years [n]	Eroded area[2] [ha]	Length of bankline [m]	Average erosion [m/year]
1988-1992	4	201.6	3.225	156
1992-1993	1	308.2	4.800	642
1993-1996	3	167.9	3.500	160
1996-1999	3	126.4	3.500	120
1988-1999	11	804.1		**191**

[1] starting from March each year

[2] estimated from measured bankline development

Table 6.3-3: Estimated annual bankline erosion rates at Kamarjani

To compute the fictitious eroded area in each subsequent year it has been assumed that a bankline equal to the extent of the structure b parallel to the bank is protected, sheltering the hinterland at an angle of 45°, starting from upstream and downstream termination of the structure respectively. Further

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it has been presumed that the fictitious annual eroded area A_e is increasing with an increasing period of time (n: number of years) considered as described by the following equation:

$$A_e = b \cdot e + (2n-1) \cdot e^2$$

This is a rather optimistic approach (in terms of prevention), which might have to be discussed in greater detail. However, consideration will be given to the uncertainty of predicted erosion rates within the sensitivity analysis.

6.3.3.2 Loss of Property

The loss of land including all houses and infrastructure, has been valued by a survey which was carried out after completion of construction. The weighted average land price in the Kamarjani area was calculated at a rate of 123,550 Tk/ha. This value appears to be quite low when compared to other studies on land prices in rural Bangladesh (e.g. FAP 20) but which is obviously justified by the comparatively lowly populated areas and the interrelated low accumulation of values (houses, infrastructure, etc.). The yearly prevented damage on land resources is calculated by multiplying the fictitious annual eroded area with the average land price. The escalation of land prices in connection with the structure implementation has not been included in the analysis, but an economic growth rate of 4% is applied for future development which is in accordance with the FPCO - Guidelines.

6.3.3.3 Loss in Agricultural Production

The benefits in terms of agricultural production generated by the Groyne Structure are limited to the prevented losses which would have incurred through crop failure in the "without case". For quantifying this effect, based on survey data, an average annual net added value gained from agricultural production of 45,000 Tk/ha (at constant year 2000 prices) was adopted in the calculation. The yearly prevented damage on crop harvest is calculated by multiplying the fictitious cumulative eroded area with the average net added value per hectare. As for the land prices, an economic growth rate of 4% is applied for future development.

6.3.3.4 Prevented Damage of BWDB Main Embankment and Manos Regulator

The prevented damage/destruction of existing river embankments was introduced in the CBA by an extra benefit of 6.12 million Tk every 5 years (refer to Subsection 6.2.7.3). Moreover, the protection of the newly built Manos regulator (1997) was taken into account with a total loss of 9.4 million Tk (source: BWDB) in year 9 after completion of the test structure.

6.3.4 Economic Feasibility Analysis (Total Investment Costs Pilot Structure)

Calculations of the Net Present Value (NPV) of Economic Costs (constant year 2000 prices) of the Kamarjani Test Structure, made by using the discounting formula under 12% discount rate, are presented in Table 6.3-4. Under consideration of all costs involved in the realization of the Kamarjani Test Site, the Economic Internal Rate of Return (EIRR) is slightly higher when compared to the assumed discount rate of 12%. Over the scheduled project period of 30 years the financial break-even point is exceeded when assuming the cost/benefit scenario described herein. Hence, the NPV is positive at a value of approx. 8.7 million Tk. In terms of protective measures, this is a considerably convincing result. Moreover, the risk of a breach at the western bank of the Jamuna, allowing the river to flood into the Ghagot river valley, would have devastating consequences in monetary terms and intangibles but these were not considered in this assessment.

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Economic Analysis of Cash Flows: Kamarjani Groyne Test Structure (Test Site I)

(constant year 2000 prices, total investment costs for pilot structure)

Assumptions				
structure length: 1,700 m	land price:	123,550.00 Taka / ha	(survey data & interviews)	
annual erosion rate: 191 m	agr. benef.:	45,000.00 Taka / ha/ year		
annual increase of benefits: 4%	4%		**EIRR :**	12.13%
			NPV (12%):	8,716,750 Tk

Period	Project costs		Prevented Damage				Constant Prices	Constant Prices
Year	Investment [1]	M&M [2]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [3]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
1993	80,609,568	0.00					-80,609,568	-71,972,829
1994	225,042,084	0.00					-225,042,084	-179,402,171
1995	139,124,296	0.00	36.12	4,640,887	1,690,327		-132,793,082	-94,519,493
1996	145,266,655	0.00	43.41	5,801,526	3,871,001		-135,594,129	-86,172,520
1997	14,251,744	0.00	50.71	7,047,590	6,592,750		-611,403	-346,927
1998	10,941,207	0.00	58.01	8,384,058	9,910,143		7,352,995	3,725,256
1999	3,794,899	0.00	65.30	9,816,166	13,881,842	6,120,000	26,023,109	11,771,533
2000		4,383,800	72.60	11,349,429	18,570,862		25,536,491	10,313,761
2001		4,383,800	79.90	12,989,647	24,044,851		32,650,698	11,774,169
2002		4,383,800	87.19	14,742,924	30,376,386		40,735,510	13,115,744
2003		4,383,800	94.49	16,615,679	37,643,288	9,400,000	59,275,166	17,040,194
2004		4,383,800	101.78	18,614,665	45,928,946	6,120,000	66,279,811	17,012,377
2005		4,383,800	109.08	20,746,986	55,322,675		71,685,860	16,428,549
2006		4,383,800	116.38	23,020,109	65,920,081		84,556,389	17,301,913
2007		4,383,800	123.67	25,441,886	77,823,455		98,881,541	18,065,288
2008		4,383,800	130.97	28,020,573	91,142,187		114,778,960	18,722,935
2009		4,383,800	138.26	30,764,849	105,993,202	6,120,000	138,494,250	20,170,904
2010		4,383,800	145.56	33,683,833	122,501,424		151,801,458	19,740,199
2011		4,383,800	152.86	36,787,113	140,800,268		173,203,580	20,110,109
2012		4,383,800	160.15	40,084,760	161,032,151		196,733,111	20,394,685
2013		4,383,800	167.45	43,587,360	183,349,044		222,552,604	20,599,383
2014		4,383,800	174.75	47,306,033	207,913,046	6,120,000	256,955,279	21,235,429
2015		4,383,800	182.04	51,252,460	234,896,995		281,765,655	20,790,912
2016		4,383,800	189.34	55,438,911	264,485,113		315,540,223	20,788,454
2017		4,383,800	196.63	59,878,274	296,873,682		352,368,156	20,727,460
2018		4,383,800	203.93	64,584,084	332,271,768		392,472,052	20,612,950
2019		4,383,800	211.23	69,570,553	370,901,974	6,120,000	442,208,727	20,736,750
2020		4,383,800	218.52	74,852,606	413,001,244		483,470,050	20,242,537
2021		4,383,800	225.82	80,445,910	458,821,706		534,883,816	19,995,709
2022		4,383,800	233.12	86,366,914	508,631,565		590,614,678	19,713,492
Total	**619,030,453**		**3,769**	**981,835,784**	**4,284,191,975**	**40,000,000**	**4,586,169,906**	**8,716,750**

[1] refer to Table A2 (Atachm.1)
[2] Monitoring & Maintenance during implementation phase included in investment costs
[3] Costs for main embankment/ Manos regulator

Table 6.3-4: Economic cash flow for the Kamarjani Groyne Field (total investment costs pilot structure, constant year 2000 prices)

6.3.5 Economic Feasibility Analysis (Adjusted Investment Costs Pilot Structure)

The assumptions made for the CBA scenario considering adjusted investment costs are briefly described in the following.

6.3.5.1 Land Acquisition

Generally it can be assumed that the groyne structure would be installed in front of an already existing main embankment. Therefore, the amount of capital used for land acquisition was reduced by the costs for the area occupied by the main embankment and by costs which occurred for land during the adaptation works (in total: approx. 2.63 million Tk).

6.3.5.2 Equipment, Construction and Adaptation

The costs related to adaptation works were interpreted as "lessons learned" and, therefore, considered as necessary costs due to the pilot nature of the Project. It is assumed that apart from the supplementary measures, adaptation of the original structure can be avoided in future projects by increasing the construction costs by 10%. Besides this, the costs for equipment procurement, hire and expatriate operators are rather high. Hence, for future projects a prerequisite should be that these works have to be executed by equipment available at the given time in Bangladesh and operated by local staff. This would lead to an estimated saving of about 50% for equipment, port charges, etc. Furthermore the groynes should be preferably connected to an already existing embankment, so that costs for earthworks. (reduction in total: approx. 157.45 million Tk).

6.3.5.3 Maintenance

A more realistic value for maintenance and repair within the "adjusted cost" scenario is approached on the basis of annual expenditures in the range of 4% of the cover layer and toe protection costs, starting after completion of the protection structure (reduction in total: approx. 37.44 million Tk).

6.3.5.4 Consulting Work

Consulting costs for the actual construction works was set to 10% of the adjusted total sum which is standard in civil engineering projects (reduction in total: approx. 86.24 million Tk).

6.3.5.5 Benefits

The benefits were considered as described in Subsection 6.3.3.

6.3.5.6 Economic Internal Rate of Return and Net Present Value

The calculations of net present value of adjusted economic costs of the Kamarjani Test Structure (under 12% discount rate) are presented in Table 6.3-5. Under the adjusted cost scenario, a positive NPV of approx. 208.8 million Tk is obtained and the Economic Internal Rate of Return is calculated at EIRR = 16.43 %. Thus, the Kamarjani Groyne Field can be interpreted as a highly promising project in terms of the financial feasibility.

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Economic Analysis of Cash Flows: Kamarjani Groyne Test Structure (Test Site I)									
(constant year 2000 prices, adjusted investment costs for pilot structure)									
Assumptions	structure length:	1,700 m		land price:	123,550.00 Taka / ha		(survey data & interviews)		
	annual erosion rate:	191 m		agr. benef.:	45,000.00 Taka / ha/ year				
	annual increase of benefits:	4%		4%				**EIRR :**	16.43%
								NPV (12%):	208,788,998 Tk
Period	Project costs		Prevented Damage				Constant Prices	Constant Prices	
Year	Investment [1]	M&M [2]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [3]	Net Cash Flow	NPV (12%) of Net Cash Flow	
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]	
1993	41,358,131	0.00					-41,358,131	-36,926,903	
1994	163,058,142	0.00					-163,058,142	-129,988,952	
1995	114,097,655	0.00	36.12	4,640,887	1,690,327		-107,766,441	-76,706,024	
1996	4,996,058	0.00	43.41	5,801,526	3,871,001		4,676,469	2,971,981	
1997		4,383,800	50.71	7,047,590	6,592,750		9,256,540	5,252,410	
1998		4,383,800	58.01	8,384,058	9,910,143		13,910,401	7,047,442	
1999		4,383,800	65.30	9,816,166	13,881,842	6,120,000	25,434,209	11,505,144	
2000		4,383,800	72.60	11,349,429	18,570,862		25,536,491	10,313,761	
2001		4,383,800	79.90	12,989,647	24,044,851		32,650,698	11,774,169	
2002		4,383,800	87.19	14,742,924	30,376,386		40,735,510	13,115,744	
2003		4,383,800	94.49	16,615,679	37,643,288	9,400,000	59,275,166	17,040,194	
2004		4,383,800	101.78	18,614,665	45,928,946	6,120,000	66,279,811	17,012,377	
2005		4,383,800	109.08	20,746,986	55,322,675		71,685,860	16,428,549	
2006		4,383,800	116.38	23,020,109	65,920,081		84,556,389	17,301,913	
2007		4,383,800	123.67	25,441,886	77,823,455		98,881,541	18,065,288	
2008		4,383,800	130.97	28,020,573	91,142,187		114,778,960	18,722,935	
2009		4,383,800	138.26	30,764,849	105,993,202	6,120,000	138,494,250	20,170,904	
2010		4,383,800	145.56	33,683,833	122,501,424		151,801,458	19,740,199	
2011		4,383,800	152.86	36,787,113	140,800,268		173,203,580	20,110,109	
2012		4,383,800	160.15	40,084,760	161,032,151		196,733,111	20,394,685	
2013		4,383,800	167.45	43,587,360	183,349,044		222,552,604	20,599,383	
2014		4,383,800	174.75	47,306,033	207,913,046	6,120,000	256,955,279	21,235,429	
2015		4,383,800	182.04	51,252,460	234,896,995		281,765,655	20,790,912	
2016		4,383,800	189.34	55,438,911	264,485,113		315,540,223	20,788,454	
2017		4,383,800	196.63	59,878,274	296,873,682		352,368,156	20,727,460	
2018		4,383,800	203.93	64,584,084	332,271,768		392,472,052	20,612,950	
2019		4,383,800	211.23	69,570,553	370,901,974	6,120,000	442,208,727	20,736,750	
2020		4,383,800	218.52	74,852,606	413,001,244		483,470,050	20,242,537	
2021		4,383,800	225.82	80,445,910	458,821,706		534,883,816	19,995,709	
2022		4,383,800	233.12	86,366,914	508,631,565		590,614,678	19,713,492	
Total	**323,509,986**		**3,769**	**981,835,784**	**4,284,191,975**	**40,000,000**	**4,868,538,973**	**208,788,998**	

[1] refer to Table A4 (Atachm.1)

[2] Monitoring & Maintenance during implementation phase included in investment costs

[3] Costs for main embankment/ Manos regulator

Table 6.3-5: Economic cash flow for the Kamarjani Groyne Field (adjusted investment costs pilot structure, constant year 2000 prices)

6.3.6 Sensitivity Analysis (Adjusted Investment Costs Pilot Structure)

To judge the influence of the most important parameters on the economic evaluation, a sensitivity analysis has been carried out. In Table 6.3-6 changes in the estimated values for i) erosion rates and ii) land prices/ agricultural losses are considered, indicating the range of calculated EIRR. Maximum increase or decrease in benefits of 40%, respectively, will demand an erosion rate between 105 m and 185 m per year to make sure that the break-even point of the project (EIRR = 12%) is achieved.

erosion rate	Change in Benefits [%]								
[m/ year]	-40	-30	-20	-10	0	10	20	30	40
50	-0.14	0.73	1.50	2.18	**2.79**	3.35	3.87	4.34	4.79
100	5.75	6.70	7.53	8.29	**8.98**	9.61	10.21	10.76	11.29
150	9.78	10.82	11.75	12.60	**13.38**	14.11	14.79	15.43	16.04
191*)	12.48	13.61	14.63	15.56	16.43	17.22	17.98	18.69	19.37
200	13.03	14.18	15.21	16.16	**17.04**	17.86	18.63	19.37	20.06
250	15.85	17.11	18.26	19.31	**20.29**	21.21	22.08	22.91	23.71

*) basic assumption for Kamarjani Groyne Field

Table 6.3-6: Sensitivity analysis of EIRR [%] (adjusted investment costs pilot structure)

In other words, under the cost assumptions made, a 40% increase of the expected benefits would justify a structure implementation even at considerably lower annual erosion rates of about 110 m. Furthermore, at the estimated erosion rate of 191m/ year, even in case of a 40 % decrease in the expected benefits, the structure would allow for an economically viable solution.

6.4 ECONOMIC EVALUATION OF THE REVETMENT AT BAHADURABAD

6.4.1 Preliminary Remarks

The Revetment Test Structure (Test Site II) is located in short distance (approx. 400 m) downstream from the Bahadurabad railway ghat on the left bank of the Jamuna river. The design of the Revetment was finalized in June 1995. Due to delays in land acquisition (socio-political reasons) and in procurement of suitable dredging equipment, the execution of work started behind schedule in December 1995, resulting in a total loss of the almost completed substructure (levee fill) of the Revetment during the monsoon season 1996, including the land which was eroded by the river (called Phase 1 hereafter). Consequently, in a second phase, additional land had to be acquired. Construction resumed in November 1996 and was completed by June 1997, protecting a bankline of about 660 m (Fig. 4.5-2). The Revetment Test Structure withstood the prolonged flood hazards of 1998 and subsequent seasons without any damage and protected the area of Kulkandi well since then.

Like Kamarjani, as revealed in the survey results, the majority of the population are farmers. However, less than forty percent have business and service as major occupation. More than 50 percent of the respondents have education up to Secondary School level.

After the completion of the Project, the cropping pattern changed to cultivation of more Irri/Boro, Aman and oilseeds. Average price of agricultural land has increased by three to four times when compared to the pre-project period.

There has been a remarkable socio-economic development in the area after the Revetment Structure was completed. About more than 5 km of brick-soiling road, a large number of pucca and semi-pucca (tin-shed) residential houses, a new market with more than 400 pucca and semi-pucca shops and other structures have been built. A new power line has been installed. Some husking mills, saw mills, new tailoring and other business shops also contributed to the substantial development of the area.

6.4.2 Assessment of Project Costs

The total capital investment for the implementation of the Revetment Structure is shown in Table 6.4-1. Costs are given in financial current prices. The site specific cost components are detailed in the following subsections.

6.4.2.1 Land Acquisition

The land required for the construction of the Revetment was financed by GoB (through FPCO/ WARPO). Since the land to be acquired in the 2. phase was of higher value and more densely populated, the amount of money required for acquisition was much larger compared to the 1. Phase (see Table 6.4-2). A sum of Tk 3,700,000 was needed as compensation money for temporary loss of crops/ harvest yields and shifting of houses in areas used for site facilities.

Item	Taka	Taka
Land acquisition/ compensation		
Land (homesteads /crop area)	7,786,923	
Roads	198,628	
Houses/ ponds/ trees/ crops	12,297,857	
Temporary land lease, etc.	3,700,000	
Administration costs	1,521,256	
Sub-Total (land acqisition/ comp.)		**25,504,664**
Consultancy		
Expatriate staff	88,034,980	
Local staff	18,392,414	
Sub-Total (consultancy)		**106,427,394**
Construction (1995 - 1997)		
Earthworks (1. phase)	23,293,353	
Earthworks (2. phase)	29,744,090	
Revetment	46,153,663	
Berm/ launching apron	105,379,345	
Berm/ falling apron	116,209,099	
Equipment procurem. and hire	29,838,158	
General[1)]	39,851,841	
Sub-Total (construction)		**390,469,550**
Adaptation/ extension		
Adaptation		
Extension		
Sub-Total (adaptation/ extension)		
Maintenance		
Monitoring	18,984,812	
Maintenance/ repair	9,151,328	
Sub-Total (maintenance)		**28,136,140**
Grand Total (investment costs)		**550,537,748**

1) Costs for site facilities, camp, port charges, insurance, etc.

Table 6.4-1: Total capital investment for the Bahadurabad Revetment (financial current prices)

6.4.2.2 Construction Costs

The earthworks in the 1. Phase and the final construction of the Revetments at Bahadurabad (2. phase) were carried out by a local sub-contractor in the construction windows 1995/96 and 1996/97,

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respectively. Besides the total loss of the earth dam during the first phase, additional unplanned costs were related to the exceptionally long construction period requiring two construction windows. Furthermore, in the second phase, it was decided to use heavy mechanical construction equipment instead of manpower to increase the progress of the works and to ensure completion before start of monsoon.

Item	Quantity [1]		Rate (Tk)	Amount (Tk)
Land acquisition				
1. Phase				
Homestead	8.89	acre	76,120.00	676,707
High Land	19.75	acre	71,235.00	1,406,891
Low Land	5.31	acre	50,800.00	269,748
Roads	2.26	acre	50,800.00	114,808
Sub-Total (a)	36.21	acre	-	2,468,154
Comp. market rate			50% of sub-total (a)	1,234,077
Sub-Total (A)				**3,702,231**
Houses	-		-	489,593
Other structures	-		-	
Ponds	-		-	1,012,847
Trees	-		-	
Crops	50.12		14,945.00	749,043
Sub-Total (B)				**2,251,483**
Sub-Total (A-B)				**5,953,714**
Add. Admin. Cost			7.5% of sub-total (A-B)	446,529
Sub-Total (1. phase)				**6,400,243**
2. Phase				
Homestead	22.03	acre	76,120.00	1,676,924
High Land	13.80	acre	71,235.00	983,043
Low Land	2.20	acre	50,800.00	111,760
Roads	1.65	acre	50,800.00	83,820
Sub-Total (c)	39.68	acre		2,855,547
Comp. market rate			50% of sub-total (c)	1,427,773
Sub-Total (C)				**4,283,320**
Houses	-		-	4,957,441
Other structures	-		-	2,266,719
Ponds	-		-	195,067
Trees	-		-	2,413,493
Crops	14.30		14,945.00	213,654
Sub-Total (D)			-	**10,046,374**
Sub-Total (C-D)			-	**14,329,694**
Add. Admin. Cost			7.5% of sub tot. (C-D)	1,074,727
Sub- Total (2. phase)				**15,404,421**
Total (land acquisit.)				**21,804,664**
Compensation costs				
land lease, etc. [2]	-		(lump sum)	3,700,000
Grand Total	**75.89**			**25,504,664**

[1] 1 acre = 0.405 ha

[2] For temporary use during implementation, incl. house shifting/ loss of crops, etc

Table 6.4-2: Land acquisition costs at Bahadurabad Test Site

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6.4.2.3 Adaptation Costs
At Bahadurabad, no adaptation of the original design was necessary.

6.4.2.4 Monitoring Costs
As mentioned in Subsection 6.3.1.4, monitoring costs were allocated appropriately to both Kamarjani and Bahadurabad test sites at a rate of 50% each. Future monitoring costs were introduced at a yearly rate of Tk 1,820,000.

6.4.2.5 Maintenance and Repair
Minor maintenance and repair works were needed at two sections of the revetment structure after the occurrence of smaller deficiencies occurred during the monsoon season of 1997. Future maintenance costs were estimated at a rate of Tk 6,061,300 per year corresponding to 4% of the cost of armour layer and launching apron.

6.4.3 Assessment of Benefits

6.4.3.1 Prevention of Potential Losses
The procedure followed for the assessment of benefits is the same as applied for Kamarjani. In Table 6.4-3 the average erosion rates for different time periods on the basis of bankline surveys between 1989 and 1999 are given. For computation of prevented losses an average weighted annual erosion rate of e = 123m per year was employed.

Period of time[1]	Number of years [n]	Eroded area[2] [ha]	Length of bankline [m]	Average erosion [m/year]
1989-1993	4	203.0	3,270	155
1993-1999	6	231.3	3,825	101
1989-1999	10	434.3		**123**

[1] starting from march each year

[2] estimated from measured bankline development

Table 6.4-3: Estimated annual bankline erosion rates at Bahadurabad

6.4.3.2 Loss of Property
The loss of land, including all houses and infrastructure has been estimated by the post implementation survey in the Bahadurabad area. The weighted average land price in the Bahadurabad area was calculated at a rate of 207,564 Tk/ha. The yearly prevented damage on land resources is determined by multiplying the fictitious annual eroded area with the average land price. The escalation of land prices in connection with the structure implementation has not been included in the analysis but an economic growth rate of 4% is applied for future development which is in accordance with the FPCO - Guidelines.

6.4.3.3 Loss in Agricultural Production
Because of the different crop pattern, slightly higher annual returns from agricultural production have been considered for Bahadurabad as compared to Kamarjani. A rate of 52,000 Tk/ha and year (at constant year 2000 prices) has been used in the calculation of the yearly prevented damage on crop harvest. An economic growth rate of 4% is applied for future development.

6.4.3.4 Prevented Damage of BWDB Main Embankment

The prevented damage/destruction of existing river embankments as described in Subsection 6.2.7.3 was introduced in the CBA by an extra benefit of approx. 3,1 million Tk every 8 years.

6.4.4 Economic Feasibility Analysis (Total Investment Cost)

Calculations of Net Present Value of Economic Costs of the Bahadurabad revetments made by using the discounting formula under 12% discount rate are presented in Table 6.4-4.

Economic Analysis of Cash Flows: Bahadurabad Revetment Test Structure (Test Site II)

(constant year 2000 prices, total investment costs for pilot structure)

Assumptions structure length: 660 m; annual erosion rate: 123 m; annual increase of benefits: 4%; land price: 207,550.00 Taka / ha; agr. benef.: 52,000.00 Taka / ha/ year; agr. benef. increase: 4% (survey data & interviews)

EIRR : 6.78%

NPV (12%): -222,644,895 Tk

Period	Project costs		Prevented Damage				Constant Prices	Constant Prices
Year	Investment [1]	M&M [2]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others[3]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
1994	63,044,108	0.00					-63,044,108	-56,289,382
1995	149,766,825	0.00					-149,766,825	-119,393,196
1996	152,724,422	0.00					-152,724,422	-108,706,227
1997	133,873,628	0.00	9.63	2,078,849	520,839		-131,273,940	-83,426,962
1998	16,574,558	0.00	12.66	2,841,253	1,253,526		-12,479,779	-7,081,362
1999	4,937,349	0.00	15.68	3,661,323	2,220,982		944,957	478,745
2000	187,489	0.00	18.71	4,542,453	3,447,897		7,802,861	3,529,618
2001		7,881,300	21.73	5,488,215	4,960,841		2,567,756	1,037,074
2002		7,881,300	24.76	6,502,370	6,788,392		5,409,462	1,950,706
2003		7,881,300	27.79	7,588,876	8,961,260		8,668,836	2,791,133
2004		7,881,300	30.81	8,751,899	11,512,429	3,096,000	15,479,028	4,449,851
2005		7,881,300	33.84	9,995,822	14,477,300		16,591,821	4,258,707
2006		7,881,300	36.86	11,325,255	17,893,844		21,337,799	4,890,073
2007		7,881,300	39.89	12,745,050	21,802,769		26,666,518	5,456,498
2008		7,881,300	42.91	14,260,307	26,247,686		32,626,693	5,960,775
2009		7,881,300	45.94	15,876,394	31,275,297		39,270,391	6,405,852
2010		7,881,300	48.97	17,598,951	36,935,586		46,653,237	6,794,780
2011		7,881,300	51.99	19,433,910	43,282,021		54,834,631	7,130,673
2012		7,881,300	55.02	21,387,508	50,371,772	3,096,000	66,973,979	7,776,133
2013		7,881,300	58.04	23,466,299	58,265,937		73,850,936	7,655,888
2014		7,881,300	61.07	25,677,173	67,029,786		84,825,659	7,851,430
2015		7,881,300	64.10	28,027,371	76,733,012		96,879,083	8,006,331
2016		7,881,300	67.12	30,524,502	87,450,003		110,093,206	8,123,553
2017		7,881,300	70.15	33,176,559	99,260,126		124,555,386	8,205,971
2018		7,881,300	73.17	35,991,942	112,248,026		140,358,668	8,256,361
2019		7,881,300	76.20	38,979,473	126,503,944		157,602,117	8,277,391
2020		7,881,300	79.22	42,148,419	142,124,052	3,096,000	179,487,172	8,416,796
2021		7,881,300	82.25	45,508,514	159,210,810		196,838,024	8,241,464
2022		7,881,300	85.28	49,069,979	177,873,335		219,062,014	8,189,255
2023		7,881,300	88.30	52,843,547	198,227,799		243,190,046	8,117,179
Total	**521,108,378**		**1,322**	**569,492,213**	**1,586,879,273**	**9,288,000**	**1,463,281,207**	**-222,644,895**

[1] refer to Table A6 (Atachm.1)   [2] Monitoring & Maintenance during implementation phase included in investment costs

[3] Costs for main embankment

Table 6.4-4: Economic cash flow for the Bahadurabad Revetment (total investment costs pilot structure, constant year 2000 prices)

Taking into consideration all costs involved in the implementation of the revetment structure, the Economic Internal Rate of Return is quite low (EIRR = 6.78%) when compared to the assumed discount rate of 12%. Over the scheduled project period of 30 years the financial break-even point is not reached and a potential loss of 222.6 million Tk. occurs.

6.4.5 Economic Feasibility Analysis (Adjusted Investment Costs Pilot Structure)

Despite the fact that the level of economic viability will not change considerably under the adjusted cost scenario, the results will be presented for the reason to provide complete information. The assumptions made for this CBA scenario are briefly described in the following.

6.4.5.1 Land Acquisition
The amount of capital used for land acquisition during the first project phase was removed, not being representative for a regular project implementation (in total: approx. 3.7 million Tk).

6.4.5.2 Equipment, Construction and Adaptation
The costs for equipment procurement, hire and expatriate construction advisers will be reduced in future projects since these works will be executed by equipment available (then) in Bangladesh and by local staff. This leads to an estimated saving of about 50% for equipment and port charges, etc. (reduction in total: approx. 15.02 million Tk).

6.4.5.3 Maintenance
A more realistic value for maintenance and repair within the "adjusted cost" scenario is approached on the basis of annual expenditures in the range of 4% of the cost of cover layer and toe protection, starting after completion of the protection structure (reduction in total: approx. 37.44 million Tk).

6.4.5.4 Consulting Work
Future consulting cost was set to 10% of the adjusted total sum which is standard in civil engineering projects (reduction in total: approx. 75.1 million Tk).

6.4.5.5 Benefits
The benefits were considered as in Subsection 6.4.3

6.4.5.6 Economic Internal Rate of Return and Net Present Value
The calculations of net present value of adjusted economic costs of the Bahadurabad revetments (under 12% discount rate) are shown in Table 6.4-5.

The economic internal rate of return under the adjusted cost scenario is still far from being sufficient (EIRR = 8.18 %). Either the price of land (and the annual benefits) or the threat of bankline erosion is too low to place priority on the protection of Bahadurabad under this cost scenario. The limiting parameters to achieve economic viability are briefly discussed in Subsection 6.4.5. However, it has to be kept in mind that due to the pilot character of the project the many structure sections consisting of several structural components (RENO© mattresses, sand bags, concrete blocks, bricks, etc.) have led to exceptionally high construction costs. Further achievable reductions in project costs will be discussed in Chapter 9.

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Economic Analysis of Cash Flows: Bahadurabad Revetment Test Structure (Test Site II)

(constant year 2000 prices, adjusted investment costs for pilot structure)

Assumptions structure length: 660 m; annual erosion rate: 123 m; annual increase of benefits: 4%; land price: 207,550.00 Taka / ha; agr. benef.: 52,000.00 Taka / ha/ year; 4%; (survey data & interviews)

EIRR : 8.18%
NPV (12%): -136,838,454 Tk

Period	Project costs		Prevented Damage				Constant Prices	Constant Prices
Year	Investment [1]	M&M [2]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [3]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
1994	43,742,757	0.00					-43,742,757	-39,056,033
1995	99,889,128	0.00					-99,889,128	-79,631,002
1996	120,228,203	0.00					-120,228,203	-85,576,060
1997	126,251,025	0.00	9.63	2,078,849	520,839		-123,651,337	-78,582,660
1998	6,338,498	0.00	12.66	2,841,253	1,253,526		-2,243,720	-1,273,147
1999		7,881,300	15.68	3,661,323	2,220,982		-1,998,995	-1,012,753
2000		7,881,300	18.71	4,542,453	3,447,897		109,050	49,329
2001		7,881,300	21.73	5,488,215	4,960,841		2,567,756	1,037,074
2002		7,881,300	24.76	6,502,370	6,788,392		5,409,462	1,950,706
2003		7,881,300	27.79	7,588,876	8,961,260		8,668,836	2,791,133
2004		7,881,300	30.81	8,751,899	11,512,429	3,096,000	15,479,028	4,449,851
2005		7,881,300	33.84	9,995,822	14,477,300		16,591,821	4,258,707
2006		7,881,300	36.86	11,325,255	17,893,844		21,337,799	4,890,073
2007		7,881,300	39.89	12,745,050	21,802,769		26,666,518	5,456,498
2008		7,881,300	42.91	14,260,307	26,247,686		32,626,693	5,960,775
2009		7,881,300	45.94	15,876,394	31,275,297		39,270,391	6,405,852
2010		7,881,300	48.97	17,598,951	36,935,586		46,653,237	6,794,780
2011		7,881,300	51.99	19,433,910	43,282,021		54,834,631	7,130,673
2012		7,881,300	55.02	21,387,508	50,371,772	3,096,000	66,973,979	7,776,133
2013		7,881,300	58.04	23,466,299	58,265,937		73,850,936	7,655,888
2014		7,881,300	61.07	25,677,173	67,029,786		84,825,659	7,851,430
2015		7,881,300	64.10	28,027,371	76,733,012		96,879,083	8,006,331
2016		7,881,300	67.12	30,524,502	87,450,003		110,093,206	8,123,553
2017		7,881,300	70.15	33,176,559	99,260,126		124,555,386	8,205,971
2018		7,881,300	73.17	35,991,942	112,248,026		140,358,668	8,256,361
2019		7,881,300	76.20	38,979,473	126,503,944		157,602,117	8,277,391
2020		7,881,300	79.22	42,148,419	142,124,052	3,096,000	179,487,172	8,416,796
2021		7,881,300	82.25	45,508,514	159,210,810		196,838,024	8,241,464
2022		7,881,300	85.28	49,069,979	177,873,335		219,062,014	8,189,255
2023		7,881,300	88.30	52,843,547	198,227,799		243,190,046	8,117,179
Total	**396,449,611**		**1,322**	**569,492,213**	**1,586,879,273**	**9,288,000**	**1,572,177,374**	**-136,838,454**

[1] refer to Table A8 (Atachm.1) [2] Monitoring & Maintenance during implementation phase included in investment costs
[3] Costs for main embankment

Table 6.4-5: Economic cash flow for the Bahadurabad Revetment (adjusted investment costs pilot structure, constant year 2000 prices)

6.4.6 Sensitivity Analysis (Adjusted Investment Costs Pilot Structure)

Due to the comparatively high capital investment for the Revetment Test Structure at Bahadurabad, a 50% increase of the annual erosion rate or a strong economical development of the protected area (more than 50% in value in terms of production and assets) would have to exist to allow for a pure economical justification of the Project.

Erosion rate	Change in Benefits [%]								
[m/ year]	-40	-30	-20	-10	0	10	20	30	40
50	-4.66	-3.60	-2.71	-1.95	-1.28	-0.68	-0.14	0.35	0.81
100	2.81	3.75	4.57	5.30	5.96	6.56	7.12	7.64	8.12
123[*)]	5.06	6.01	6.85	7.61	8.18	8.52	9.50	10.04	10.55
150	7.29	8.28	9.16	9.94	10.66	11.32	11.94	12.51	13.06
200	10.74	11.80	12.76	13.62	14.41	15.14	15.82	16.47	17.07
250	13.65	14.80	15.87	16.78	17.65	19.46	19.21	19.93	20.60

*) basic assumption for Bahadurabad Revetments

Table 6.4-6: Variation in EIRR [%] under different assumptions regarding erosion rates and benefits (adjusted investment costs pilot structure)

6.5 ECONOMIC EVALUATION OF THE REVETMENT AT GHUTAIL

6.5.1 Preliminary Remarks

Ghutail is situated about 4 km downstream from the Bahadurabad ghat. The construction works started in December 1999 and were completed by June 2000. The crest length of the embankment structure is 604 m protecting a bankline of about 550 m (Fig. 4.7-1).

Like Kamarjani and Bahadurabad, Ghutail also has an agriculture based economy. Irri/Boro and Aman are the major crops with wheat, sugarcane and jute as minor crops. However, as Ghutail is a big market place and business centre, about forty percent respondents reported business and service as their major occupation.

Ghutail is the most important ghat for the exchange of goods between the zones of Bogra and Jamalpur/Sherpur area. In Ghutail relatively intense economic activities existed which concentrated on the trans-shipment of goods from country boats to trucks for further transport to Islampur and Jamalpur. Connection to the national power grid exists in this place since 1997. The economic value of the land and structures of the bazar is rather high.

After the commencement of the works of the bank protection structure many new pucca/semi-pucca houses, shops, masjid and schools have been built. There has been a tremendous increase in price of land since the construction started. Increased economic activities will result in further benefits to the people of the locality.

6.5.2 Assessment of Project Costs

The total capital investment employed in the implementation of the revetment structure is shown in Table 6.5-1. Costs are given in financial current prices. The site specific cost components are described in detail in the following Subsections.

6.5.2.1 Land Acquisition

The land required for the construction of the Revetment was financed by GoB (through FPCO/ WARPO). Weighted average land prices for the mouzas Gilabari and Belgacha are given (see Table 6.5-2). A sum of Tk. 595,430 was needed for land lease (compensation money for temporary loss of crops/ harvest yields and shifting of houses in areas used for site facilities).

Item	Taka	Taka
Land acquisition/ compensation		
Land (homesteads /crop area)	1,141,252	
Roads		
Houses/ ponds/ trees/ crops	5,972,313	
Temporary land lease, etc.	595,430	
Administration costs	533,517	
Sub-Total (land acqisition/ comp.)		**8,242,513**
Consultancy		
Expatriate staff	33,692,400	
Local staff	7,039,072	
Sub-Total (consultancy)		**40,731,472**
Construction (1999 - 2000)		
Earthworks	10,303,629	
Revetment	31,322,307	
Berm/ launching apron	18,586,680	
Berm/ falling apron	84,272,660	
General[1]	14,416,887	
Sub-Total (construction)		**158,902,163**
Adaptation/ extension		
Extension		
Adaptation		
Sub-Total (adaptation/ extension)		
Maintenance		
Monitoring	1,394,683	
Maintenance/ repair		
Sub-Total (monitoring/ maintenance)		**1,394,683**
Other		
Road	7,025,375	
Sub-Total (other)		**7,025,375**
Grand Total (investment costs)		**216,296,206**

[1] Costs for site facilities, camp, port charges, insurance, etc.

Table 6.5-1: Total capital investment for the Ghutail Revetment

6.5.2.2 Construction Costs

The construction costs are reasonably lower when compared with the other project locations. This is due to the lower hydraulic priority class used as a basis for the structure design and due to the application of only two typical revetment designs.

6.5.2.3 Adaptation Costs

At Ghutail Test Site, an adaptation of the original design was not necessary until now.

Construction on the flood plain is the preferred option for any type of impermeable groyne. However, the cofferdams can also be built in shallow water with moderate flow. This is not possible for an earth dam or a concrete spur.

The time needed to mobilise the site is short, provided that the materials can be taken from a stockpile or another source nearby. Construction time is similar for all considered and is generally not a critical parameter.

If necessary, the earthdam can be adapted relatively easily to changing local conditions. This does not apply for cofferdams and concrete spurs. Participation of local population and labour force in construction activities is best suited for labour intensive earthworks, such as main embankments, and casting of concrete block for slope and toe protection.

7.4.3 Revetments and Bed Protections

This subsection summarizes the experience gained with different protection systems, which were used and tested within this Project. The experience mainly results from the monitoring of the Revetment Test Structures at Bahadurabad and Ghutail but also the observations made at the Groyne Test Structure at Kamarjani (impermeable groyne sections, bed protections around groyne piles and revetments of main embankment) have influenced the evaluation.

General qualities of the tested materials for bank protection works are compiled in Table 7.4-3. Some supplementary notes regarding their applicability are given in the following.

With the exception of some specific geotextile fabrics (articulating and collapsing mattresses), all materials are available in Bangladesh or could be reproduced within the country.

A high flexibility of protection systems is of advantage with respect to the revetment's adaptability to potential settlements of the soil and subsequent deformation of the structure. Launching aprons shall be highly flexible to follow potential erosion of the riverbed near the bankline and stabilise thereby the developing under water slope. High flexibility of filter materials furthermore reduces the risk of damages during installation of the cover material.

Revetments must provide a sufficient permeability to avoid uplift forces from groundwater pressure. A high permeability of the revetment is also favourable from the environmental point of view, as it avoids a separation of the terrestrial and aquatic biospheres.

The requirements for maintenance and repair are in an acceptable range for all investigated systems. Generally, all systems need regular monitoring and immediate repair in case of observed damages, to avoid further destruction. Slope protections by Durba grass sods are particularly vulnerable to damages by human intervention or animals and therefore need more intensive observation and nursing during the first years. However, repair is easy and can be done with local labour at low cost.

The classification of the materials with respect to their environmental suitability followed a general assessment. Durban grass and Vetiver are natural materials and have therefore been classified with the highest environmental suitability. They are followed by rip-rap which has an almost natural-like appearance and encourages development of marine life and plants to settle in the voids between the individual units. Cover layers with a continuous geotextile fabric were ranked with a low environmental suitability, as they separate the subsoil from the aquatic zone.

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S.-No.	Structure Type	Origin	Flexi-bility	Permea-bility	Mainte-nance Require-ments	Environmental Impact	
						Visual	Ecological
1. Rock / Boulders							
1.1	Rip-rap	Bangladesh	High	High	Low to Medium	Acceptable to Good	Acceptable to Good
1.2	Hand pitched stone	Bangladesh	Low	Low	Medium	Acceptable to Good	Poor to Acceptable
1.3	Cement grouted stone	Bangladesh	Low	Low	Low to Medium	Poor to Acceptable	Poor
1.4	Bitumen grouted stone	Bangladesh	Medium	Low to Medium	Low	Poor to Acceptable	Poor to Acceptable
2. Gabions							
2.1	Mattresses (brick or stone fill)	Reproducible in Bangladesh	High	High	Low to Medium	Acceptable	Acceptable
2.2	Box Gabions (stone/rock fill)	Reproducible in Bangladesh	Medium	High	Low to Medium	Poor to Acceptable	Acceptable
2.3	Gabion Sacks (stone/rock fill)	Reproducible in Bangladesh	Medium to High	High	Low to Medium	Acceptable	Acceptable
3. Concrete (pre-cast units)							
3.1	CC-slabs	Bangladesh (site production)	Medium	Low	Low	Poor	Poor
3.2	CC Interlocking slabs	Bangladesh (factory make)	Medium	Low	Low	Poor	Poor
3.3	Hand pitched CC-blocks	Bangladesh (site production)	Medium	Low	Low	Poor	Poor
3.4	Dumped CC-blocks	Bangladesh (site production)	High	Medium to High	Low	Poor to Acceptable	Poor to Acceptable
4. Articulating Mattresses							
4.1	Gabion mattresses, steel wire linked (with stone fill)	Reproducible in Bangladesh	High	High	Medium	Acceptable	Acceptable
4.2	CC-blocks attached to geotextile filter mat, steel wire linked	In-situ cast, geotextile to be imported	High	Medium	Medium	Acceptable	Poor
4.3	Tubular geotextile fabric mattress: sand or bitumen-sand filled	To be imported	High	Medium	Medium	Poor	Poor to Acceptable
4.4	Collapsible geotextile block mattress, sand filled	To be imported	High	Medium	Medium	Poor	Poor to Acceptable
4.5	Collapsible Concrete Block Mattress	To be imported	Low	Low	Low to Medium	Poor	Poor
5. Sand Containers							
5.1	Geotextile-Sand bags (up to 250 kg)	Reproducible in Bangladesh	Medium	Low to Medium	Low to Medium	Poor	Poor
5.2	Geotextile-Sand containers (up to 900 kg)	Reproducible in Bangladesh	Medium	Low to Medium	Low to Medium	Poor	Poor
6. Bio-Engineering							
6.1	Durba grass sods on Geo-Jute Soil Saver	Bangladesh	High	High	High	Good	Good
6.2	Vetiver plantation	Bangladesh	High	High	Medium	Good	Good

Table 7.4-3: Multi-criteria analysis of materials for protective layers for standard erosion protection measures (Loading Classes 2 and 3)

The applicability of the protective systems with regard to hydraulic loads and other criteria is summarised in Table 7.4-4 and discussed thereafter.

Well dimensioned rip-rap layers can be utilized for slope and bed protections even under high flow velocity attack. Installation under water is generally possible but requires special methods to suit the conditions prevalent at and in the Jamuna river. For short bank area and for the repair of existing revetments, hand pitched stones are a suitable system. For high hydraulic loads and slopes steeper than 1V : 2H, grouting is recommended to increase the stability.

S. No.	Structure Type	Loading Class				Bank Slope			Main Applicability
		1	2	3	4	<1:2 V:H	> 1:2 V:H	Near Vertical	
		+ = Recommended				- = Not recommended			
1. Rip-rap									
1.1	Rip-rap	+	+	+	-	+	-	-	• bank and bed protection • installation above and below water level
1.2	Hand pitched stone	+	+	+	-	+	+	-	• short bank reaches • repair of existing revetments • installation above water level
1.3	Cement grouted stone	+	+	+	+	+	+	-	• areas of attack by strong currents • installation above water level
1.4	Bitumen grouted stone	+	+	+	+	+	+	-	• areas of attack by strong currents • installation above water level
2. Gabions									
2.1	Mattresses (brick or stone fill)	+	+	+	+	+	-	-	• bank protection of large areas • installation above water level
2.2	Box Gabions (stone/rock fill)	+	+	+	+	+	+	+	• retaining wall for bank protection • installation above water level
2.3	Gabion Sacks (stone/rock fill)	+	+	+	+	+	-	-	• toe protection • installation above and below water level
3. Concrete (pre-cast units)									
3.1	CC-slabs	+	+	-	-	+	+	-	• slope protection • installation above water level
3.2	CC-Interlocking slabs	+	+	-	-	+	+	-	• slope protection • installation above water level
3.3	Hand pitched CC-blocks	+	+	+	+	+	+	+	• slope protection • installation above water level
3.4	Dumped CC-blocks	+	+	+	+	+	+	-	• bed and bank protection in case of strong current and wave attack • installation above and below water level

Table 7.4-4: Applicability of materials for revetments and bed protections
(table continued next page)

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S. No.	Structure Type	Loading Class				Bank Slope			Main Applicability
		1	2	3	4	<1:2 V:H	> 1:2 V:H	Near Vertical	
		+ = Recommended				- = Not recommended			
4. Articulating Mattresses									
4.1	Gabion mattresses, steel wire linked (with stone fill)	+	+	+	-	+	+	-	• launching apron • slope protection • construction above water level (limited water depths) [1]
4.2	CC-blocks attached to geotextile filter mat, steel wire linked	+	+	+	-	+	+	-	• launching apron • slope protection • construction above water level (cast in place) [1]
4.3	Tubular geotextile fabric mattress: sand filled or bitumen-sand filled	+	+	-	-	+	+	-	• launching apron • slope protection • installation above water level [1]
4.4	Collapsible sand filled geotextile mattress	+	+	-	-	+	+	-	• launching apron • slope protection • installation above water level [1]
4.5	Collapsible concrete filled geotextile mattress	+	+	+	-	+	+	-	• launching apron • slope protection • installation above water level [1]
5 Sand Containers									
5.1	Geotextile-Sand bags (up to 250 kg)	+	-	-	-	+	+	+	• falling apron and toe protection • installation above and below water level
5.2	Geotextile-Sand containers (up to 900 kg)	+	+	+	-	+	+	+	• falling apron and toe protection • installation above and below water level
6. Bio-Engineering									
6.1	Durba grass sods	+	-	-	-	+	-	-	• upper reaches of banks above mean water level • preferably on land-side • on river-side prone to wave erosion • installation above water level
6.2	Vetiver plantation	+	+	-	-	+	-	-	• toe protection to upper reaches of banks • installation above water level

Note: 1) Articulating mattress systems can generally be installed below water level, but require special floating equipment that is presently not available in Bangladesh. With the prevalent flow conditions construction of under-water slope protections should be avoided for erosion protection structures.

Table 7.4-4 (continued): Applicability of materials for revetments and bed protections

Gabions are a very effective protection system. They consist of wire mesh boxes which are filled in-situ with bricks or natural stones. Box gabions can also be pre-cast and transported to the site which, however, requires heavy equipment. Locally produced wire mesh gabions showed significant signs of corrosion and insufficient strength and integrity. It is recommended to use only wire mesh of high quality, preferably machine made with PVC-waling.

Hand pitched cc-blocks are the recommended protection system for the slopes of standard revetment structures above the water line. Concrete blocks are more durable than bricks and less prone to pilferage. If well dimensioned, they are able to suit any flow condition. Sizes of 30-50 cm have proven to be sufficient to withstand the hydraulic loads occurring at the Jamuna river.

Articulating mattresses have been used as launching aprons for the protection of the toe of revetments. They are preferably installed on the flood plain, but can also be installed in shallow water, using purpose made steel frames. The mattresses must be at the revetment's toe. Gabion-mattresses and concrete block mattresses can be produced largely from local materials.

Geotextile sand containers are an easy-to-install system and can be tailored locally. They can be used for falling aprons and as toe protection but are also very suitable for strategic stockpiling, which makes them a suitable material for immediate repair measures. Installation is possible above and below water level.

For slopes with minor flow attack, natural protection by Durba grass or Vetiver is the favourable solution. Durba grass is preferably used for the land-sided slope of main embankments or for the upper area of river-sided slopes. Due to stronger roots, Vetiver is a suitable material for toe protections.

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8 OVERVIEW OF RECENT BANK PROTECTION STRUCTURES IN BANGLADESH

8.1 INTRODUCTION

Flood protection work has a long history in Bangladesh. The main activities of BWDB initially concentrated on the establishment of embankments along the rivers to provide flood protection to mainly agricultural areas. Such embankments can be built at relatively low cost and with local materials and capacities only. However, as these structures are built without any slope or toe protection they are not suitable to withstand the erosive forces of the major rivers. Consequently, the embankments are easily eroded, if once attacked by the river. In such cases, immediate repair measures are carried out, consisting mainly in the dumping of concrete blocks or other materials. After the monsoon season, the damaged embankments get repaired or replaced by a new embankment which is built in the hinterland with a sufficient safety distance to the river (so-called retired embankments).

The design philosophies of **bank protection** works deals with the establishment of structures These are strong enough to resist the hydraulic loads not only during flood discharge but also throughout the whole year. Such structures require suitably designed revetments and toe protections and hence high initial investment costs which are mostly provided by Donors, whereas the maintenance and adaptation works are executed under GoB responsibility. However, due to the lack of funds, maintenance is often not possible to the required extent.

A major improvement in comparison to previous structures is the use of geotextile filters. After lifting of the import ban in 1994, such materials allow the construction of more effective revetments and bed protections.

Establishment of major bank protection works in Bangladesh were started with the investigation program for the Jamuna bridge. Construction activities in Bangladesh executed during the last 10 years concentrate on the Brahmaputra / Jamuna river. A number of major bank protection works were built since then with a total investment cost of about US$ 450 Million.

8.2 SELECTED RECENT PROJECTS

8.2.1 General Remarks

In the following, major structures built along the Jamuna river within the last 10 years are presented briefly. Practically, all executed works represent isolated individual solutions built for the first time in this scale in Bangladesh. According to their specific tasks and the local conditions, the type and design of the structures differ significantly.

The locations of the major protection works at the Jamuna are shown in Fig 8.2-1 and a compilation of their main features is given in Table 8.2-1, which also contains some projects executed at other major rivers in Bangladesh. The assessment of the related costs is described in Subsection 8.2.3.

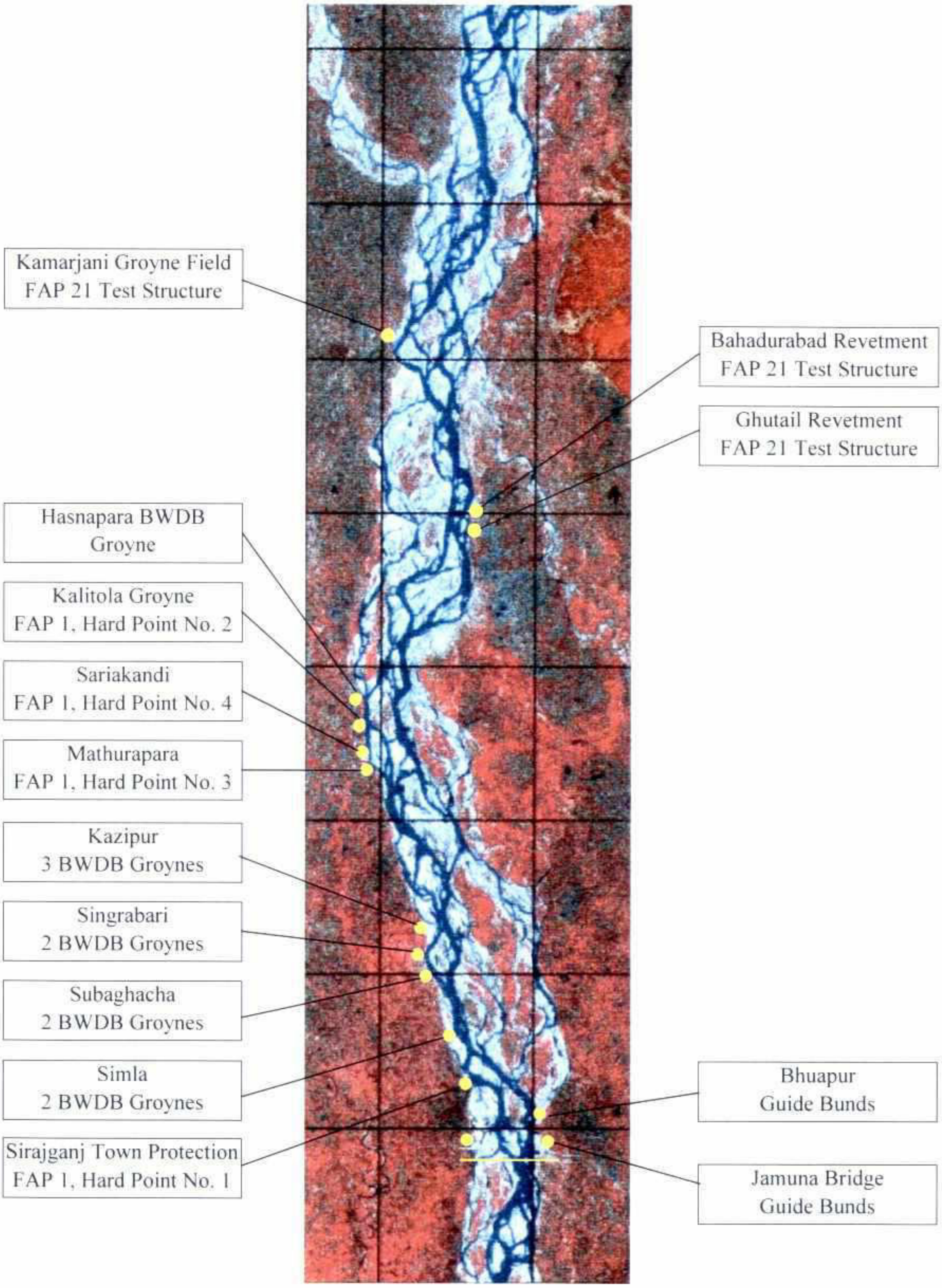

Fig. 8.2-1: Location of recent major bank protection works at the Jamuna river (construction periods are given in Table 8.2-1)

Project	Location		Description			Period		Investment			Executing
	River	Location	Type	Size	Protected Length	From	To	Local	Foreign	Financing Body	Authority
Jamuna Bridge River Training Works											
Jamuna Bridge Guide Banks	Jamuna	Sirajganj Bhuapur	Revetment (Rip rap)	2 x 3,200 m	2 x 3.200 m	1994	1998	not known	288 Mio US$	IDA, ADB, Japan	JMBA
Jamuna Bridge Bhuapur	Jamuna	Bhuapur	Revetment (cellular geotextile mat)	1.700 m	1,550 m	1994	1995	not known	6.8 Mio US$	IDA, ADB, Japan	JMBA
FAP 1 (Hard Points at BRE)											
Sirajganj Town Protection	Jamuna	Sirajganj	Revetment	2.650 m	2.450 m	1996	1998	not known	73.5 Mio US$	IDA	BWDB
Sariakandi Hard Point	Jamuna	Sariakandi Area	T-Head Groyne	700 m (revetment)	4,500 m	1996	1999	not known	21.1 Mio US$	IDA	BWDB
Mathurapara Hard Point	Jamuna		T-Head Groyne	650 m (revetment)		1996	1999	not known	20.1 Mio US$		BWDB
Kalitola Groyne	Jamuna		Impermeable Groyne	ca 400 m		1996	1998	not known	10.4 Mio US$		BWDB
FAP 21 Pilot Project											
Kamarjani Groyne Test Structure	Jamuna	Kamarjani	7 permeable groynes	120 m main groynes	1.700 m	1994	1995	79.8 Mio Tk	17.9 Mio DM	KfW, Germany AFD, France	FPCO / MoWR
Bahadurabad Revetment Test Structure	Jamuna	Bahadurabad	Revetment	950 m	670 m	1996	1997	50.8 Mio Tk	15.8 Mio DM	KfW, Germany AFD, France	WARPO / MoWR
Ghutail Revetment Test Structure	Jamuna	Ghutail	Revetment	600 m	425 m	1999	2000	14.5 Mio Tk	6.6 Mio DM	KfW, Germany AFD, France	WARPO / MoWR
Local Structure Designs											
Impermeable Groynes at Meghai near Kazipur	Jamuna	Kazipur Area	3 Impermeable RC-Groynes	variabel	ca. 3,000 m	1999	2000	ca. 220 Mio Tk	n.a.	n.a.	BWDB
Impermeable Groynes at Suboghacha	Jamuna	Suboghacha	2 Impermeable RC-Groynes	variabel	ca. 1,400 km	1999	2000	ca. 80 Mio Tk	n.a.	n.a.	BWDB
Impermeable Groynes at Simla	Jamuna	Simla	2 Impermeable RC-Groynes	variabel	not known	1999	2000	ca. 60 Mio Tk	n.a.	n.a.	BWDB
Impermeable Groynes at Singrabari	Jamuna	Singrabari	2 Impermeable RC-Groynes	variabel	not known	1998	1999	25 Mio Tk per Groyne according to information obtained from BWDB during field trip in 1999			BWDB
Hasnapara Bank Protection Scheme	Jamuna	Hasnapara	Impermeable Groyne	ca. 150 m	n.a.	1996	1998	not known	not known	-	BWDB
Protection to Pabna Irrigation Project	Jamuna	Kiotala Pumpstation	Slope Revetment	2.000 m	2,000 m	2000	2001[1)]	160 Mio Tk (estimate)	n.a.	n.a.	BWDB
Impermeable Groynes downstream of Teesta Barrage	Teesta	Shailmari and Kilkandi near Dalia	Impermeable RC-Groynes	RC-part 150 m earth dam variable	n.a	1999	2001[1)]	300 Mio Tk (estimate)	n.a.	n.a.	BWDB
Rajshahi Town Protection	Ganges	Rajshahi	Groyne Rehab Revetment Solid Groyne	3,500 m 60 m RC	not known	1999	ongoing	300 Mio Tk (estimate)	not known	IDA	BWDB
Protection work at Charghat	Ganges	Charghat of Rajshahi	Bank Slope Rrevetment	1,050 m	not known	1997	2001[1)]	540 Mio Tk each (PP costs)	n.a.	n.a.	BWDB
Protection to Ganges Kabadak Pumpstation & Intake channel	Ganges	Downstream of Harding-Bridge	Bank Slope Rrevetment	700 m (revetment)	not known	1997	1999	180 Mio Tk	n.a.	n.a.	BWDB
Protection of Paka-Narayanpur	Ganges	Near Nawabgonj	8 T-Head Groynes	250-750 m (groyne heads)	not known	2001	2002[1)]	750 Mio Tk	n.a.	n.a.	BWDB
Protection to Meghna-Dhanagoda Project	Meghna	Mohanpur and Ekhlaspur	Bank Slope Rrevetment	not known	not known	2000	2001[1)]	160 Mio Tk (estimate)	n.a.	n.a.	BWDB

Table 8.2-1: Recent bank protection and river training measures in Bangladesh

8.2.2 Description of Structures

(a) Jamuna Multipurpose Bridge

Since the eighties of the 20th century the lower Jamuna / Brahmaputra river has been more closely studied due to plans to build a multi purpose bridge across the river. The concept consisted of encroaching the river to a width of about 4.5 km, to limit the length of the bridge. For this purpose, 3 kilometer long guide bunds were built on both banks of the river upstream and downstream of the structure. Further upstream protective works gradually decrease the average width of the river from about 11 km to 5 km towards the bridge. To prevent outflanking of the bridge at its eastern side an about 1.5 km long revetment was built at Bhuapur, about 5 km upstream.

The guide bunds of the Jamuna bridge were built under controlled still water conditions, i.e. in a trench, which was dredged behind the bankline (on the eastern side) and on a char (on the western side) respectively. Substantial earth fill was required to connect the western guide bund to the river's bank line and to fill the area of the bank protection work above design flood level.

In consequence of severe slides, which occurred during dredging of the under water slope (design slope 1V : 3.5H) at the western side, the slope was modified to 1V : 6H, this has remained stable until now. The guide bunds were protected by a revetment of stone rip-rap placed on a geotextile filter with a falling apron as toe protection. Above water, open stone asphalt was used as cover layer.

The structures were completed in 1998 and no damages have been reported since then.

Worth mentioning in terms of efforts, cost and problems was the resettlement plan for a vast number of people living in the affected area. Before the start of the construction work, the area was the most densely populated in Bangladesh maybe for the reason that the settlers expected compensation for their land.

(b) River Bank Protection Project (formerly FAP 1: Brahmaputra Right Embankment)

Under the Flood Action Plan, component 1: Brahmaputra Right Embankment, the most erosion prone locations on the right bank were identified and a list of priority measures was determined. A generalized concept for the protection of erosion prone reaches, based on so-called 'hard points', was proposed.

The following structures were built within FAP 1 up to the year 2000:

Efforts to control or limit erosion of the Jamuna / Brahmaputra river in the area of Sirajganj have a long history, starting as early as the 70ies of the 20th century and culminating in the construction of the Ranigram groyne immediately upstream from Sirajganj. In 1998, **Sirajganj Town Protection** (Hard Point No. 1) was built to protect the township of Sirajganj against flooding and to prevent the river from outflanking the Jamuna Bridge. Sirajganj is located about 5 km north of the bridge at the western river bank (Fig. 8.2-1).

The new structure was built in 1998 and consists of a 2.3 km long earth dam which starts at the Ranigram groyne and joins the main embankment at its downstream end (Fig. 8.2-2). The enclosed water area behind the new structure was filled with sand for the purpose of resettlement.

Typical sections of the revetment are shown in Fig. 8.2-3.

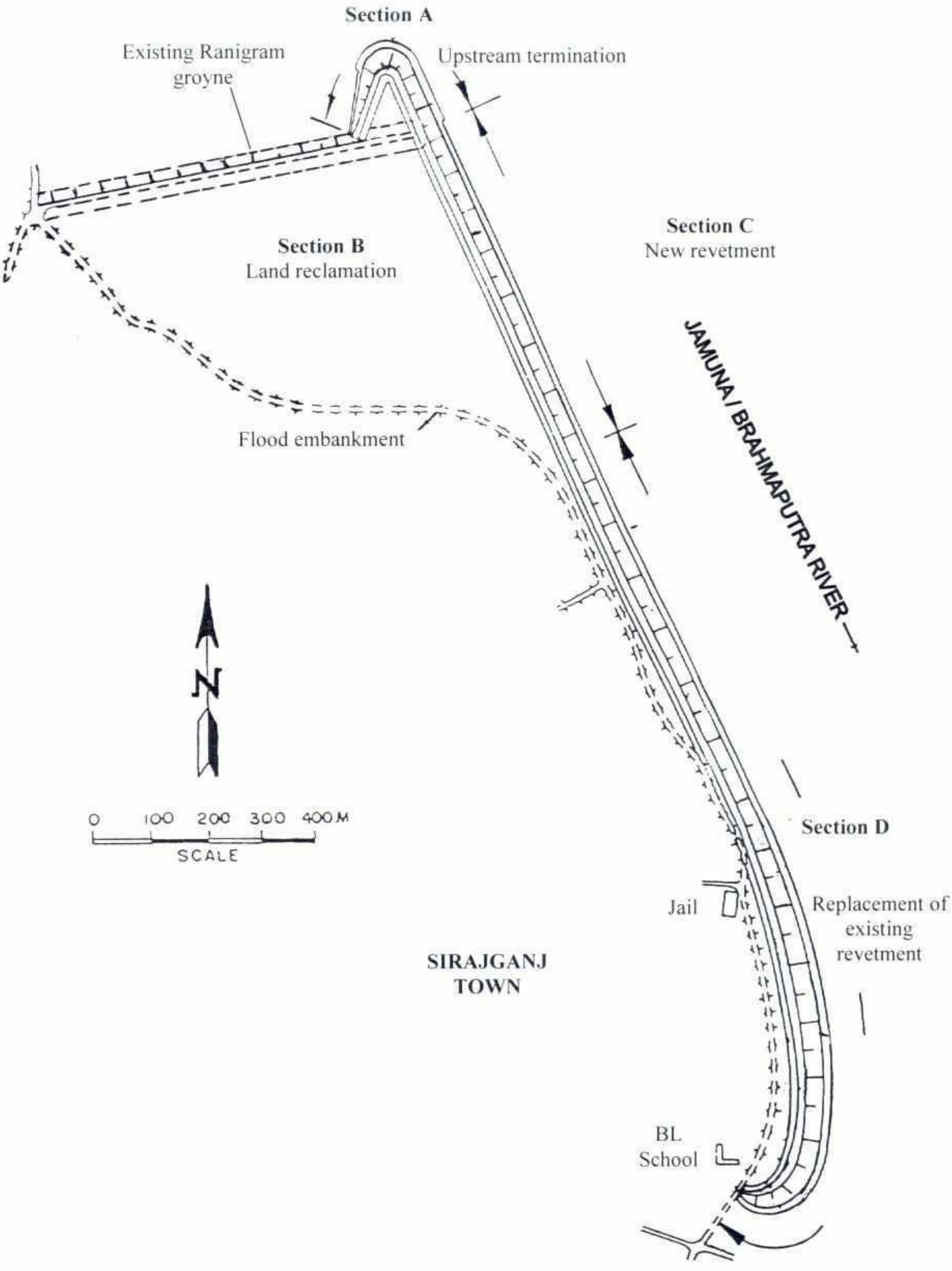

Fig. 8.2-2: Layout of FAP 1 hard point No. 1 at Sirajganj

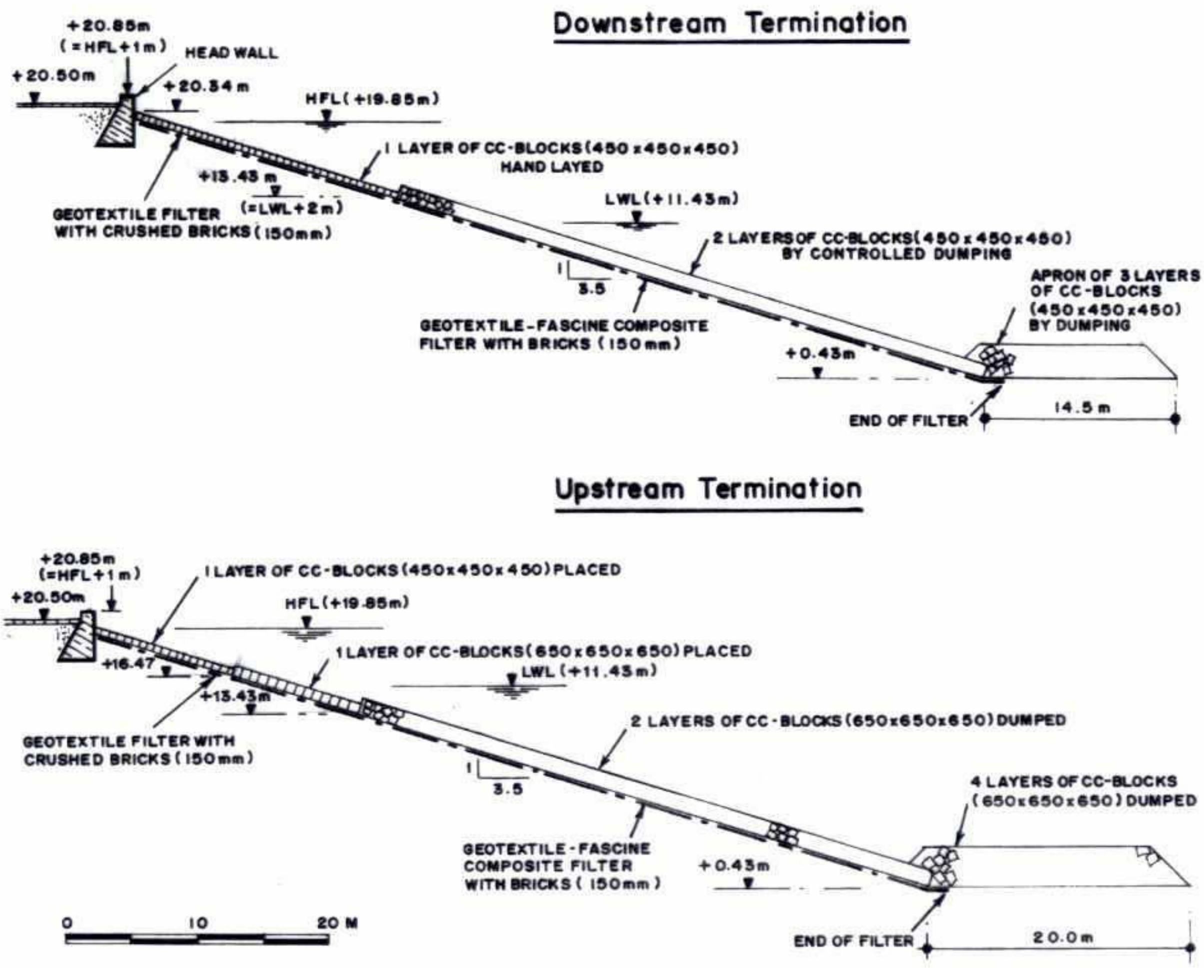

Fig. 8.2-3: Typical cross sections of FAP 1 hard point No. 1 (Sirajganj)

Due to its purpose to function as a town protection, the structure was built at a high safety level. The crest rises 1 m above the design flood level (related to the discharge of 1% exceedance). The outer face of the dam is sloped 1V : 3.5H and protected by concrete blocks size 0.45 x 0.45 x 0.45 m^3 (in the upper part) and 0.65 x 0.65 x 0.65 m respectively (in the lower part). The blocks were placed on geotextile filters and covered by brick chips. Below low water level, the slope protection consists of two layers of concrete blocks sized 0.65 x 0.65 x 0.65 m which were dumped on a composite geotextile filter with bamboo grid fascine and brick fill. The slope ends at 0.43 m+PWD. The toe of the dam was secured by a 20.5 m wide falling apron of dumped concrete blocks.

Initial dredging of the slope profile was carried out in still water conditions, protected by a char, which had formed in front of the town.

During the monsoon flood of 1998 a 26 m deep scour hole developed at the upstream termination of the structure, resulting in a total water depth of nearly 50 m. This scour was caused by vortices created by a flow separation at the structures' head. The falling apron could not protect the developing under water slope sufficiently. As a consequence, several slides occurred in the area of hydraulic fill and lead to the destruction of the upstream part of the revetment. Permanent dumping of geobags and concrete blocks during the flood season could reduce but not totally stop the erosion.

After the flood, the revetment was repaired. The under water slopes had to be profiled under flow conditions leading to substantial problems in maintaining the foreseen levels. The riverbed around the upstream termination was protected by flexible mattresses over a width of up to 100 m from the toe.

Monitoring during the 1999 flood showed a new scour at the downstream end of the these mattresses which caused minor damages to the bed protection.

Photo 8.2-1: Damaged revetment at Sirajganj with dumped concrete blocks (situation in October 1998)

Mathurapara and **Sariakandi Hard Points** (No. 3 and 4) were built in 1998 about 30 km upstream from the Jamuna Bridge on the western river bank to prevent the Jamuna river from breaking into the Bengali river and eventually outflanking the new bridge (Fig. 8.2-4). They were designed as groynes with a horseshoe shaped head with a distance of about 2 km between each other.

Each groyne consists of a 500 m long revetment (the groyne head)m which runs parallel to the river and is connected to the main embankment by an earthen dam which prevents overland flow. The groyne heads were built immediately at the bankline, with their outer faces sloping down to the riverbed. Underwater dredging of the slope was carried out under moderate flow conditions. The cross-sections through the groyne heads are similar to those of the "Hard Point" at Sirajganj (Fig. 8.2-3).

The connecting earth dams were completely built on the floodplain. Their slopes are protected by brick mattresses (Photo 8.2-2).

Erosion of the area between the groynes during the monsoon flood(s) was expected and led to progressive development of the groyne characteristics.

During the flood of 1998, the riverbed at the upstream termination of the Mathurapara Hard Point eroded to such an extent that the falling apron of the groyne head could not protect the developing underwater slopes sufficiently and was therefore replenished by dumped concrete blocks. Downstream and upstream from the groyne, the floodplain eroded as a consequence of vortices, caused by flow separations at the groyne heads (Photo 8.2-3). Due to the progressively increasing protrusion of the groynes into the river, the total water depth at the upstream termination of Mathurapara Hard Point had developed up to about 30 meters by June 2001, that is, the scour hole extended below the apron setting level.

No serious damages of the structure at Sariakandi have been observed since then. Upstream from the groyne, the floodplain eroded whereas downstream sediment was deposited.

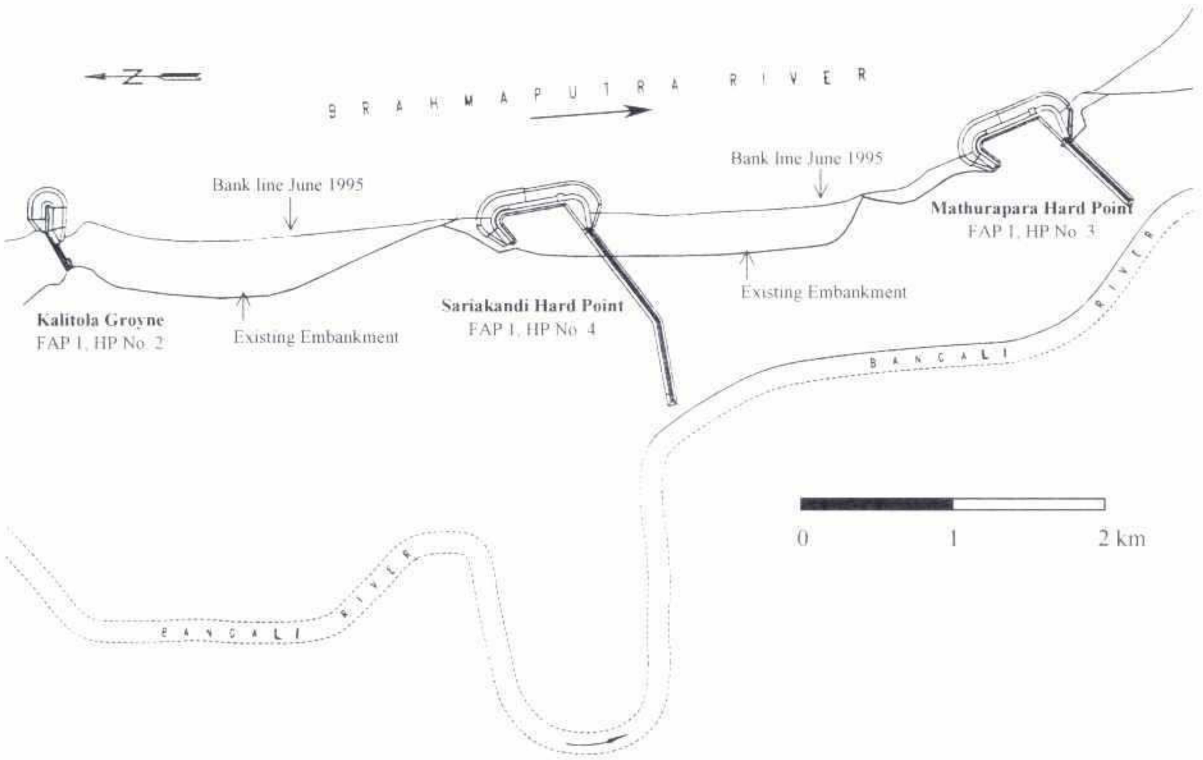

Fig. 8.2-4: General layout plan of FAP 1 hard points No. 2, 3 and 4 at the Sariakandi area

Photo 8.2-2: Earthdam with brick mattresses at "hard point" No. 3. The main embankment can be seen in the background (situation in December 1999)

Photo 8.2-3: Erosion downstream from "hard point" No. 3 at Mathurapara (situation in December 1999)

The **Kalitola Groyne** is located about 2 km north of Sariakandi Hard Point (Fig. 8.2-4). A first structure had already been built in 1988. After several emergency repairs the groyne was rebuilt and strengthened by BWDB in 1997/98. The general design of the 110 m long head of the groyne corresponds to that of the revetments at Sirajganj, Mathurapara and Sariakandi. The groyne was covered with a concrete block revetment and linked to the existing embankment by a 200 m long earth dam, which itself is protected by brick mattresses placed on a geotextile filter.

Downstream from the groyne strong erosion of the bank occurred. The mechanism is similar to that observed at the end of the impermeable groyne sections of the Kamarjani Test Structure. A flow separation at the groyne head caused strong return currents downstream from the groyne, these lead to erosion of the riverbed. During the flood of 1999, a 16 m deep scour hole developed at the head of the groyne and caused a partially failure of the revetment by slope slides (Fig. 8.2-5). The damage was repaired by the dumping of concrete blocks and sand filled geotextile bags (Photo 8.2-4).

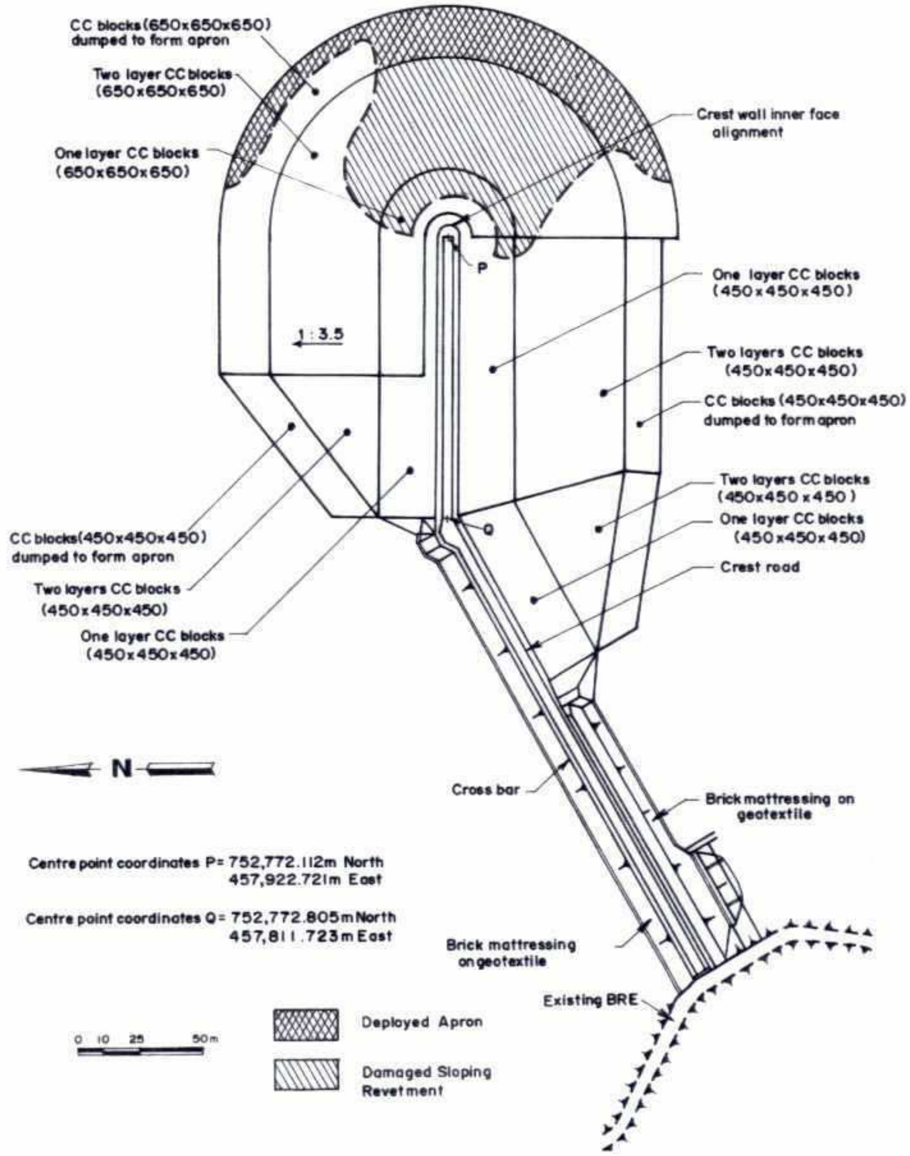

Fig. 8.2-5: Layout of Kalitola Groyne showing damages which occurred in 1999

Photo 8.2-4: Head of Kalitola Groyne with dumped concrete blocks and sand bags (situation in December 1999)

(c) FAP 21: Bank Protection Pilot Project

Component 21 of the Flood Action Plan concentrated on the development of cost effective bank protection structures for Bangladeshi rivers. The implemented structures are described in detail in Chapter 3 (**Kamarjani Groyne Field**) and 4 (**Revetment Test Structures** at **Bahadurabad** and **Ghutail**) of this Report. They are included in Table 8.2-1 for the sake of completeness.

(d) BWDB-Groynes

In the last years, several impermeable groynes have been built by BWDB on the right bank of the Jamuna between Singrabari and Kazipur, namely

- 3 groynes at Meghai
- 2 groynes at Singrabari
- 2 groynes at Subogacha
- 2 groynes at Simla

Another groyne at Simla is under construction since recently and two further groynes at Betil and Enaetpur are planned.

The purpose of the groynes is to provide bank protection to areas identified during the FAP 1 study as vulnerable to erosion. In a first step, BWDB concentrated its activities on the area between Sirajganj and Sariakandi.

The groynes consist of a 150 m long impermeable concrete structure (the groyne head) which is connected to the main embankment by an earth dam (Photos 8.2-5 and 8.2-6). The groyne heads are made of concrete piles with an interlocking concrete wall and a deck slab on top. They were built on the dry floodplain. Around the groyne heads, the floodplain has been protected over a width of about 20 m by a falling apron of concrete blocks.

The connecting earth dams were also built on the floodplain, with exception of the groyne at Meghai, where the dam crosses a near-bank channel which was cut-off. The heads of the dams were protected by hand-pitched concrete blocks installed on a geotextile filter.

Photo 8.2-5: BWDB-groyne at Singrabari. In the background the earthdam (situation in December 1999)

Photo 8.2-6: BWDB-groyne at Singrabari. The protection around the concrete structure has partially displaced (situation in December 1999)

Item	Quantity [1]		Rate (Tk) [2]	Amount (Tk)
Land acquisition				
Homestead	5.65	acre	68,266.50	385,706
High Land	0.97	acre	77,118.50	74,805
Low Land	8.74	acre	34,362.00	300,324
Roads				
Sub-Total (a)	15.36	acre	-	760,835
Comp. market rate			50% of sub-total (a)	380,417
Sub-Total (A)				**1,141,252**
Houses	-		-	4,352,754
Other structures	-		-	26,400
Ponds	-		-	935,334
Trees	-		-	378,525
Crops	-		-	279,300
Sub-Total (B)	-		-	**5,972,313**
Sub-Total (A-B)	-			**7,113,565**
Add. Admin. Cost	-		7.5% of sub tot. (A-B)	533,517
Total (land acquisit.)				**7,647,083**
Compensation costs				
Land lease, etc.[3]	-		(lump sum)	595,430
Total				**8,242,513**

[1] 1 acre = 0.405 ha

[2] Total quantities/ weighted average rates for mouzas Gilabari and Belgacha are given

[3] For temporary use during implementation, including house shifting/ loss of crops, etc.

Table 6.5-2: Land acquisition costs at Ghutail Test Site

6.5.2.4 Monitoring Costs

As mentioned previously, monitoring costs were mostly shared between both Kamarjani and Bahadurabad test sites at a rate of 50% each. Only minor costs for operation and for repair of existing equipment were needed. Future monitoring costs were introduced at an annual rate of Tk 1,820,000.

6.5.2.5 Maintenance and Repair

Future maintenance costs were estimated at a rate of Tk 1,996,400 per year.

6.5.3 Assessment of Benefits

6.5.3.1 Prevention of Potential Losses

The procedure followed for the assessment of benefits is the same as applied for the other project sites. In Table 6.5-3, the average erosion rates for different time periods are given on the basis of bankline surveys between 1989 and 1999. In the computation of prevented losses an average weighted annual erosion rate of e = 196 m per year was employed.

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Period of time[1]	Number of years [n]	Eroded area[2] [ha]	Length of bankline [m]	Average erosion [m/year]
1989-1993	4	203.0	3.285	341
1993-1999	6	200.0	3.370	99
1989-1999	10	403.0		**196**

[1] starting from March each year

[2] estimated from measured bankline development

Table 6.5-3: Estimation of annual bankline erosion rates at Ghutail

6.5.3.2 Loss of Property

The loss of land including all houses and infrastructure has been valued by applying the rates from post project surveys. The weighted average land price in the area of Ghutail was calculated at a rate of 506,048 Tk/ha. The yearly prevented damage on land resources is calculated by multiplying the fictitious annual eroded area with average land price. The escalation of land prices in connection with the structure implementation has not been included in the analysis, but an economic growth rate of 4% is applied for future development which is in accordance with the FPCO - Guidelines.

6.5.3.3 Loss in Agricultural Production

As for Bahadurabad an agricultural output of 52,000 Tk/ha and year (at constant year 2000 prices) was used in the calculation of the yearly prevented damage on crop harvest. An economic growth rate of 4 % is applied for future development.

6.5.3.4 Prevented Damage of BWDB Main Embankment

The prevented damage/destruction of existing river embankments as described in subsection 6.2.7.3 was introduced in the CBA by an extra benefit of approx. 1.89 million Tk every 5 years.

6.5.4 Economic Feasibility Analysis (Total Investment Costs Pilot Structure)

Calculations of Net Present Value of Economic Costs of the Ghutail revetments made by using the discounting formula under 12% discount rate are presented in Table 6.5-4. Under consideration of all costs involved in the implementation of the revetment structure, the economic internal return rate is very high (EIRR = 27.46 %). Over the scheduled project period of 30 years a potential economic profit of 563 million Tk can be expected.

Economic Analysis of Cash Flows: Ghutail Revetment Test Structure (Test Site III)									
(constant year 2000 prices, total investment costs for pilot structure)									
Assumptions structure length:		550 m	land price:	506,000.00 Taka / ha			(survey data & interviews)		
annual erosion rate:		196 m	agr. benef.:	52,000.00 Taka / ha/ year					
annual increase of benefits:		4%	4%					**EIRR :**	27.46%
								NPV (12%):	563,176,894 Tk
Period	Project costs		Prevented Damage					Constant Prices	Constant Prices
Year	Investment [1]	M&M [2]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others[3]		Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]		[Tk]	[Tk]
1998	19,667,191	0.00						-19,667,191	-17,559,992
1999	62,460,770	0.00						-62,460,770	-49,793,343
2000	85,273,033	0.00	14.62	7,694,471	790,736			-76,787,826	-54,656,058
2001		3,816,400	22.30	12,207,185	2,076,859			10,467,644	6,652,377
2002		3,816,400	29.99	17,068,605	3,914,019			17,166,225	9,740,577
2003		3,816,400	37.67	22,299,408	6,362,219			24,845,226	12,587,365
2004		3,816,400	45.35	27,921,365	9,486,097	1,980,000		35,571,061	16,090,542
2005		3,816,400	53.04	33,957,399	13,355,234			43,496,233	17,567,399
2006		3,816,400	60.72	40,431,642	18,044,473			54,659,715	19,710,841
2007		3,816,400	68.40	47,369,492	23,634,263			67,187,356	21,632,530
2008		3,816,400	76.09	54,797,680	30,211,016			81,192,296	23,340,845
2009		3,816,400	83.77	62,744,332	37,867,491	1,980,000		98,775,423	25,353,191
2010		3,816,400	91.45	71,239,039	46,703,199			114,125,838	26,154,696
2011		3,816,400	99.14	80,312,933	56,824,829			133,321,362	27,280,192
2012		3,816,400	106.82	89,998,755	68,346,707			154,529,061	28,231,882
2013		3,816,400	114.50	100,330,942	81,391,264			177,905,807	29,020,291
2014		3,816,400	122.19	111,345,707	96,089,557	1,980,000		205,598,863	29,944,311
2015		3,816,400	129.87	123,081,123	112,581,792			231,846,515	30,149,226
2016		3,816,400	137.55	135,577,219	131,017,900			262,778,719	30,510,390
2017		3,816,400	145.24	148,876,073	151,558,134			296,617,806	30,749,408
2018		3,816,400	152.92	163,021,911	174,373,699			333,579,210	30,875,962
2019		3,816,400	160.60	178,061,216	199,647,428	1,980,000		375,872,244	31,063,026
2020		3,816,400	168.29	194,042,829	227,574,485			417,800,914	30,828,675
2021		3,816,400	175.97	211,018,074	258,363,117			465,564,790	30,672,388
2022		3,816,400	183.65	229,040,869	292,235,438			517,459,907	30,438,703
2023		3,816,400	191.34	248,167,859	329,428,272			573,779,732	30,135,376
2024		3,816,400	199.02	268,458,544	370,194,028	1,980,000		636,816,172	29,862,590
2025		3,816,400	206.70	289,975,414	414,801,634			700,960,648	29,348,709
2026		3,816,400	214.38	312,784,100	463,537,520			772,505,220	28,878,775
2027		3,816,400	222.07	336,953,521	516,706,655			849,843,776	28,366,021
Total	**167,400,994**		**3,314**	**3,618,777,704**	**4,137,118,066**	**9,900,000**		**7,495,351,976**	**563,176,894**

[1] refer to Table A10 (Atachm.1)
[2] Monitoring & Maintenance during implementation phase included in investment costs
[3] Costs for main embankment

Table 6.5-4 : Economic cash flow for the Ghutail Revetment (total investment costs pilot structure, constant year 2000 prices)

6.5.5 Economic Feasibility Analysis (Adjusted Investment Costs Pilot Structure)

Due to the relatively small investment costs and the previous experience gained during the two other projects no larger savings under the adjusted cost scenario can be assumed.

6.5.5.1 Land Acquisition
No reductions applied.

6.5.5.2 Equipment, Construction and Adaptation
No reductions applied.

6.5.5.3 Maintenance
No reductions applied.

6.5.5.4 Consulting Work

Future consulting cost was set to 10% of the adjusted total sum which is standard in civil engineering projects. (reduction in total: approx. 21.4 million Tk).

6.5.5.5 Benefits

The benefits were considered as in Subsection 6.4.3

6.5.5.6 Economic Internal Rate of Return and Net Present Value

The calculations of net present value of adjusted economic costs of the Ghutail revetments (under 12% discount rate) are shown in Table 6.5-5. The very high EIRR of 29.7 % (corresponding to a NPV of approx. 583 million Tk) highlights the strong priority of the Ghutail area in financial terms under the assumptions made in this assessment. In addition to the substantial erosion rates, the short duration of project realization, followed by early returns in benefits supported the high profit.

Economic Analysis of Cash Flows: Ghutail Revetment Test Structure (Test Site III)

(constant year 2000 prices, adjusted investment costs for pilot structure)

Assumptions structure length: 550 m; land price: 506,000.00 Taka / ha (survey data & interviews)
annual erosion rate: 196 m; agr. benef.: 52,000.00 Taka / ha/ year
annual increase of benefits: 4%; 4%
EIRR : 29.70%
NPV (12%): 583,220,374 Tk

Period	Project costs		Prevented Damage				Constant Prices	Constant Prices
Year	Investment [1]	M&M [2]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [3]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
1998	16,352,120	0.00					-16,352,120	-14,600,107
1999	54,734,837	0.00					-54,734,837	-43,634,277
2000	69,924,858	0.00	14.62	7,694,471	790,736		-61,439,651	-43,731,530
2001		3,816,400	22.30	12,207,185	2,076,859		10,467,644	6,652,377
2002		3,816,400	29.99	17,068,605	3,914,019		17,166,225	9,740,577
2003		3,816,400	37.67	22,299,408	6,362,219		24,845,226	12,587,365
2004		3,816,400	45.35	27,921,365	9,486,097	1,980,000	35,571,061	16,090,542
2005		3,816,400	53.04	33,957,399	13,355,234		43,496,233	17,567,399
2006		3,816,400	60.72	40,431,642	18,044,473		54,659,715	19,710,841
2007		3,816,400	68.40	47,369,492	23,634,263		67,187,356	21,632,530
2008		3,816,400	76.09	54,797,680	30,211,016		81,192,296	23,340,845
2009		3,816,400	83.77	62,744,332	37,867,491	1,980,000	98,775,423	25,353,191
2010		3,816,400	91.45	71,239,039	46,703,199		114,125,838	26,154,696
2011		3,816,400	99.14	80,312,933	56,824,829		133,321,362	27,280,192
2012		3,816,400	106.82	89,998,755	68,346,707		154,529,061	28,231,882
2013		3,816,400	114.50	100,330,942	81,391,264		177,905,807	29,020,291
2014		3,816,400	122.19	111,345,707	96,089,557	1,980,000	205,598,863	29,944,311
2015		3,816,400	129.87	123,081,123	112,581,792		231,846,515	30,149,226
2016		3,816,400	137.55	135,577,219	131,017,900		262,778,719	30,510,390
2017		3,816,400	145.24	148,876,073	151,558,134		296,617,806	30,749,408
2018		3,816,400	152.92	163,021,911	174,373,699		333,579,210	30,875,962
2019		3,816,400	160.60	178,061,216	199,647,428	1,980,000	375,872,244	31,063,026
2020		3,816,400	168.29	194,042,829	227,574,485		417,800,914	30,828,675
2021		3,816,400	175.97	211,018,074	258,363,117		465,564,790	30,672,388
2022		3,816,400	183.65	229,040,869	292,235,438		517,459,907	30,438,703
2023		3,816,400	191.34	248,167,859	329,428,272		573,779,732	30,135,376
2024		3,816,400	199.02	268,458,544	370,194,028	1,980,000	636,816,172	29,862,590
2025		3,816,400	206.70	289,975,414	414,801,634		700,960,648	29,348,709
2026		3,816,400	214.38	312,784,100	463,537,520		772,505,220	28,878,775
2027		3,816,400	222.07	336,953,521	516,706,655		849,843,776	28,366,021
Total	**141,011,815**		**3,314**	**3,618,777,704**	**4,137,118,066**	**9,900,000**	**7,521,741,156**	**583,220,374**

[1] refer to Table A12 (Atachm. 1)
[2] Monitoring & Maintenance during implementation phase included in investment costs
[3] Costs for main embankment

Table 6.5-5: Economic cash flow for the Ghutail Revetment (adjusted investment costs pilot structure, constant year 2000 prices)

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6.5.6 Sensitivity Analysis

The Ghutail Revetment plays an exceptional role with regard to the economical output. This is underlined by the results of the sensitivity analysis (Table 6.5-6), where it is demonstrated that even for 30% reduced benefits and an annual erosion rate of approx. 100 m, a reasonable economical feasibility is provided.

Erosion rate	Change in Benefits [%]								
[m/ year]	-40	-30	-20	-10	0	10	20	30	40
50	1.99	3.15	4.16	5.04	5.84	6.56	7.23	7.85	8.43
100	11.10	12.36	13.50	14.54	15.50	16.40	17.25	18.06	18.82
150	17.43	18.97	20.37	21.68	22.91	24.08	25.20	26.27	27.31
196*)	22.50	24.35	26.06	27.67	29.70	30.67	32.08	33.45	34.78
250	28.10	30.37	32.49	34.51	36.45	38.32	40.13	41.88	43.60

*) basic assumption for Ghutail Revetments

Table 6.5-6: Variation in EIRR [%] under different assumptions regarding erosion rates and benefits (adjusted investment costs pilot structure)

6.6 COMPARISON OF TEST STRUCTURES

Fig. 6.6-1 presents a re-evaluation of the different Test Sites in economical terms and demonstrates the financial priority order of the structures and the respective locations investigated. Nevertheless, it has to be considered that the comparatively low construction costs at Ghutail where mainly due to the notably lower estimates regarding the assumed hydraulic loads (i.e., followed by significantly smaller scour depth at the structure front), so that a direct comparison with the other tested structures cannot be made.

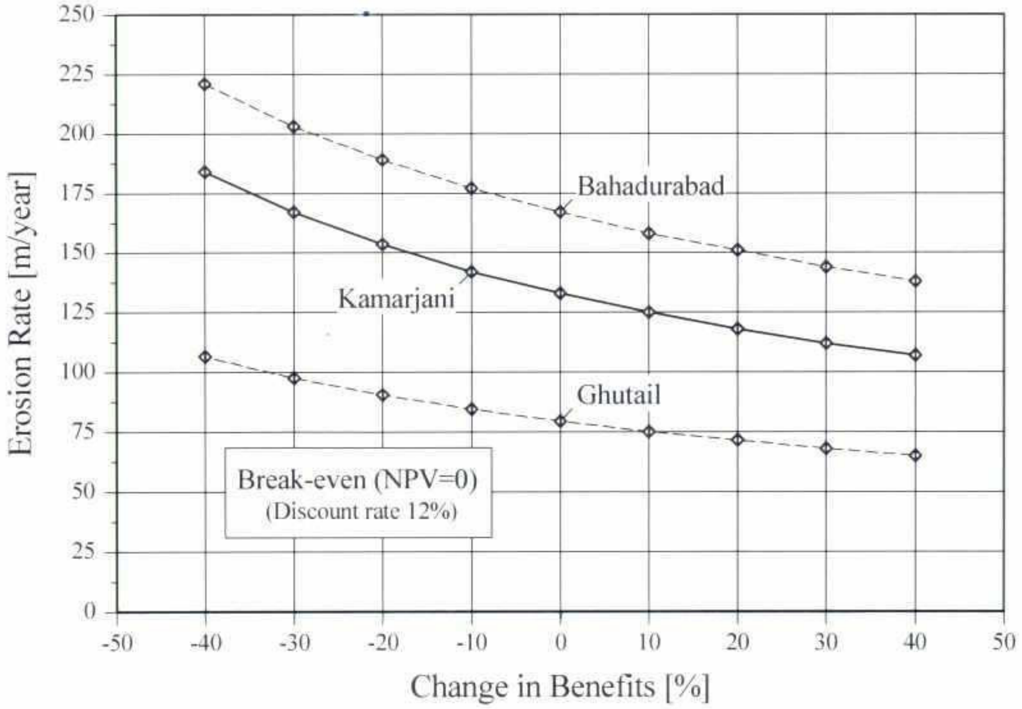

Fig. 6.6-1: Comparison of the investigated FAP 21 pilot structures in economic terms

7 MULTI-CRITERIA ANALYSIS OF TEST STRUCTURES

7.1 GENERAL

The Groyne Test Structure at Kamarjani along with the Revetment Test Structures at Bahadurabad and Ghutail have proven their general suitability to serve as an erosion protection structure under the prevailing conditions at the Jamuna river. The structures can be built at relatively low cost by combining the utilization of local materials or materials, which could be produced in the country in future years, and utilization of construction equipment and skill available in Bangladesh.

Each type of construction has not only advantages but also certain limitations. These depend on the local situation, executional constraints as well as environmental, sociological and other conditions. Therefore, no general preference can be given to any of the structures. The decision on which type of structure should be applied, must be taken under careful examination of the decisive boundary conditions for the individual case. A compilation of significant criteria for the selection of the preferable type of structure (groyne field or revetment structure) for a selected location is given in Subsection 7.3.

Different designs and construction methods were applied for the test structures and different materials were used during the Project. Thereby, certain limitations and preferences could be identified. The properties of different types of piles for permeable groynes and different designs for impermeable groyne sections are compared in Section 7.4. Furthermore, the applicability of different protective materials for slope and bed protections for the main rivers of Bangladesh and the use of these materials with respect to the expected hydraulic loads were compared.

7.2 CLASSIFICATION

For easier standardisation of future erosion protection structures, a classification with respect to the expected hydraulic loads is proposed. Accordingly, 4 loading classes were distinguished and are compiled in Table 7.2-1. If the expected loads (flow velocity, wave height, total scour depth) do not correspond to the same loading class, the applicable loading class is defined by the most unfavourable parameter.

Loading Class 1 corresponds to light and moderate flow and wave attack and applies for smaller rivers or branches with minor discharge. For such locations, erosion can be stopped by traditional low cost measures. More exposed locations are covered by the Loading Classes 2 and 3 which may be considered as the standard classes for erosion protection measures at the Jamuna and other major rivers in Bangladesh. Such areas can be protected effectively by the developed and proposed standard structures. Loading Class 4 covers exceptional hydraulic conditions, calling for specially designed protection structures. However, also for the latter Loading Class, the principles of the proposed standard design can be applied with suitable adaptation.

Loading Class		Loads			Remarks
No	Description	Depth averaged flow velocity $\bar{u}$ [m/s]	Design wave height H_{des} [m]	Total scour / water depth $h_0 + y_s$ [m]	
1	Light	< 1.0	< 0.25	< 10 m	Local measures
2	Moderate	1.0 – 2.5	0.25 - 0.5	10 – 20 m	Standard protection structures
3	High	2.5 – 3.5	0.5 - 1.0	20 – 30 m	
4	Exceptional	> 3.5	> 1.0	> 30 m	Specific measures

Table 7.2-1: Loading classes

7.3 GROYNE FIELD VERSUS REVETMENT STRUCTURE

In the following, a comparison between a groyne structure and a revetment structure is presented with respect to significant technical and socio-economical criteria. Other criteria, such as financial or environmental aspects depend largely on the local situation of the envisaged structure and can therefore not be compared in a general manner. They were discussed in previous chapters of this Report and are not included in this comparison.

The comparison is restricted to standard structures, applicable for the Loading Classes 2 and 3. It is based on the evaluation of the implemented test structures at Kamarjani, Bahadurabad and Ghutail. Nonetheless, the findings are of general validity and can be transferred to other erosion protection structures of similar type.

The evaluated significant criteria are compiled in Table 7.3-1 and have been rated qualitatively. For a quantitative evaluation, a weighing-factor must be introduced for each criteria which, however, depends on the local conditions and can therefore not be assessed generally.

A **groyne structure** in this context means a field of permeable or semi-permeable groynes which are linked to a main embankment and extend into the river. A **revetment structure** is a reinforced embankment of a certain length with protected slopes and a launching as well as a falling apron as toe protection. It should be mentioned that in some design aspects the impermeable part of a groyne corresponds more to a revetment structure than to a pile structure. Therefore, difference has been made between permeable and the impermeable groynes (or groyne sections).

Only the general suitability of the structures has been considered and evaluated. The different construction methods, materials and systems which can be employed are compared in the following section.

Principally, both types of structures can be used, even at locations with high hydraulic loads. However, a revetment structure (and similarly the impermeable part of a groyne) must be fully completed before the water level starts to rise, as the incomplete structure otherwise is prone to damage or even total failure due to undermining by river bed erosion. In contrast to this, a permeable groyne of piles can even be completed when the water level has already started to rise and will not fail completely, even if a single pile is lost.

The use of local materials and construction methods by Bangladesh contractors are possible for groynes and revetments. However, fabrication capacities for the future production of geotextile fabrics and large steel piles have to be set-up in the country. Strategic fabrication and stockpiling of groyne piles should be established. Due to the large quantities of material needed for the construction of a revetment structure, it is recommended that strategic stockpiling should be limited to key materials.

S. No.	Criteria	Groyne Structure		Revetment Structure	Remarks
		Permeable	Impermeable		
1. Hydraulic Loads					
1.1	Favourable to unexpected hydraulic loads	Yes	Moderate	Moderate	
1.2	Favourable to unexpected scouring	Yes	Limited	Limited	
2. Producibility with Local Materials and Skill					
2.1	Suitable for Bangladesh contractors	Yes		Yes	
2.2	Use of local materials	Possible	Largely	Largely	
2.3	Materials to be imported or reproduced in Bangladesh	Large diameter steel piles	Geotextile filter	Geotextile filters and mattresses	
2.4	Suitable for strategic material stock-piling	Yes	Limited	Limited	
2.5	Specialised skills	Large dia. pile installation		Mattress installation	
3. Construction Constraints					
3.1	Construction on the flood plain	Yes		Yes	
3.2	Construction into the river under moderate flow conditions	Yes	No	No	
3.3	Prone to early raise of river water level (risk of destruction of unfinished works)	No	Yes	Yes	
3.4	Site mobilisation time (the period required after award to physically start the construction)	Short [1] (Moderate)		Short	3)
3.5	Construction time	Moderate		Long	
4. Adaptability, Maintenance and Repair					
4.1	Adaptability to specific local conditions (e.g. to suit river response)	Good		Good	
4.2	Requirements for Maintenance and Repair	Low	Moderate [2] High	Moderate [2] High	
5. Socio-Economic Aspects					
5.1	Job opportunities and participation of local population during construction	Low	Moderate	High	
5.2	Land acquisition demand	Low	Moderate	High	Dependent on configuration

Notes:
1) short, if the piling equipment is available and operational, otherwise moderate
2) moderate, if the toe protection is accessible during the dry season, otherwise high
3) provided, material can be taken from strategic stock

Table 7.3-1: Multi-criteria analysis of standard erosion protection structures (for Loading Classes 2 and 3)

A revetment structure of the type favoured for future standard designs cannot be built into the water but must be built on the flood plain. Construction time for a revetment structure is extensive due to the large amount of earthworks and the time-consuming installation of slope and bed protections. Site ꝩ80

mobilization time is, however, short. In contrast to this, the site mobilization time for permeable groynes may be longer since it depends on the availability of the suitable piling equipment.

Both groynes and the revetment structures can be adapted with moderate effort to varying morphological conditions, e.g. the river response after a monsoon season. Maintenance and adaptation of the pile structures can be executed relatively with ease, even if the piles are (partially) submerged, whereas adaptation works for the impermeable sections of the groynes or revetment structures can only be carried out on land during low water periods.

Involvement of local population in construction activities is higher for revetment structures, as the earthworks and execution of revetments can be done by unskilled human labour to a large extend.

A typical FAP 21 revetment structure with its large falling and launching aprons must be built entirely on the flood plain, whereby this type of structure requires relatively extensive areas of land. The same holds for impermeable groyne sections, whereas permeable groyne structures can be built partially into the river, in which case land acquisition is significantly less.

7.4 CONSTRUCTION METHODS, MATERIALS AND SYSTEMS

7.4.1 Groyne Piles

Groyne piles of steel and concrete were tested within the Project. In Table 7.4-1 steel piles, in-situ cast reinforced concrete piles and pre-cast pre-stressed concrete piles are compared with regard to significant criteria.

Other materials usually used, like wood or plastic, are not available in the required sizes and are, therefore, not considered in the following comparison. They can, however, be used for the implementation of groynes at locations with low hydraulic loads, applicable to the Loading Class 1.

Due to their limited lengths, concrete piles are restricted to moderate water depths and should, therefore, only be applied to the inner part of the groynes. Steel piles, however, can also be employed at locations with exceptional scouring depths, as they are less vulnerable to excessive scouring, always be keeping in mind, that the embedment lengths of the piles are safely chosen.

All investigated pile systems can be produced and installed by local contractors. However, steel piles of the required sizes though not available in standard sections, can be rolled and welded on demand.

For the quick implementation between two monsoon seasons, the construction window and other construction constraints are important criteria for the selection of the preferred pile system. In this context, pre-cast piles have an advantage over in-situ cast piles. The reason being that pre-cast piles can be stockpiled and installed in the water even under moderate flow conditions, whereas in-situ concrete piles can (under the present circumstances) only be built in the dry (on the flood plain). In addition, installation time for a single pile is much shorter for pre-cast piles than for in-situ piles. On the other hand, in-situ bored piles can be installed with conventional equipment which is sufficiently available, enabling several piles to be built at once, thus reducing the overall construction time. Mobilization time for the construction of structures of pre-cast piles depends largely on the availability of the suitable installation equipment. It is recommended to provide a complete installation set through BWDB which can be hired by the local contractors.

An important advantage of steel piles is their easy adaptability, e.g. extension and redriving follow excessive scour depths.

Installation of steel piles and pre-cast pre-stressed concrete piles requires highly skilled labour and special equipment and therefore only allows for limited participation of the local population. This is contrary to in-situ cast bored concrete piles which can be built with conventional equipment and less skilled local labour.

S. No.	Criteria	Steel Piles	Concrete Piles	
			In-situ cast	Pre-cast Pre-stressed
1. Hydraulic Loads				
1.1	Vulnerable to unexpected hydraulic loads	No	No	No
1.2	Vulnerable to unexpected scouring	No	Yes	Yes
2. Producibility with Local Materials and Skill				
2.1	Suitable for Bangladesh contractors	Yes	Yes	Yes
2.2	Availability of material in Bangladesh	Reproducible	Available	Available
2.3	Suitable for strategic material stock-piling	Yes	Yes (basic materials)	Yes
2.4	Specialised skills	Pile fabrication, welding and large dia. pile installation	No	Pile fabrication,
3. Construction Constraints				
3.1	Installation on the flood plain	Limited by possible total driving depth	Yes	Limited by possible total driving depth
3.2	Installation into the river under moderate flow conditions	Yes	No	Yes
3.3	Site mobilisation time (the period required after award to physically start the construction)	Depends on availability of piling equipment	Short	Depends on availability of piling equipment
3.4	Installation time for single pile	Short	Moderate	Short
4. Adaptability, Maintenance and Repair				
4.1	Adaptability to specific local conditions (e.g. for repair or to suit river response)	Easy	Not possible, piles must be replaced	Very complicated
5. Socio-Economic Aspects				
5.1	Job opportunities and participation of local population during construction	Low	Moderate	Low

Table 7.4-1: Multi-criteria analysis of groyne piles applicable for standard erosion protection measures (for Loading Classes 2 and 3)

7.4.2 Impermeable Groyne Sections

For the impermeable part of the groynes, protected earthdams and cofferdams of sheet piles were implemented and investigated. A concrete block gravity structure would not be feasible under the prevailing subsoil-conditions at the Jamuna and was consequently not considered. Alternatively to the investigated pilot structures, BWDB has built concrete spurs at various locations along the Jamuna river. For completeness, this type of groyne was included into the comparison, though it was not part of the Project. Significant criteria are compiled in Table 7.4-2 and discussed in the following.

S. No.	Criteria	Earth Dams	Cofferdams Steel sheet piles	Cofferdams CC sheet piles	Concrete Spurs
1. Hydraulic Loads					
1.1	Creation of smooth transition between impermeable and permeable section of the groyne	Good	Limited	Limited	Not applicable
1.2	Favourable to unexpected hydraulic loads	Limited	Yes	Yes	Yes
1.3	Favourable to unexpected scouring	No	Limited [1)]	Limited [1)]	Limited [1)]
2. Producibility with Local Materials and Skill					
2.1	Availability of material in Bangladesh	Available	Available	Available	Available
2.2	Suitable for Bangladesh contractors	Yes	Yes	Yes	Yes
2.3	Suitable for strategic material stock-piling	Yes [2)]	Yes [2)]	Yes [2)]	Limited
2.4	Specialised skills	No	Sheet pile installation		No
3. Construction Constraints					
3.1	Construction on the flood plain	Yes	Yes	Yes	Yes
3.2	Construction into the river under moderate flow conditions	No	Limited	Limited	No
3.3	Site mobilization time	Short	Short	Short	Short
3.4	Construction time	Moderate	Moderate	Moderate	Moderate
4. Adaptability, Maintenance and Repair					
4.1	Adaptability to specific local conditions (e.g. for repair or to suit river response)	Easy	Complicated	Complicated	Complicated
5. Socio-Economic Aspects					
5.1	Job opportunities and participation of local population during construction	High	Good	Good	Good

Notes: 1) depending on the embedded length of the sheet piles / concrete structure
2) it is assumed, that the relatively small quantities of earth required for the construction are taken from borrow pits nearby and must therefore not be stockpiled

Table 7.4-2: Multi-criteria analysis of impermeable groyne sections for standard erosion protection structures (Loading Classes 2 and 3)

Principally, all types of impermeable groynes can be employed even under high hydraulic loads, provided that they are protected against scouring with a suitably dimensioned bed protection / toe protection and that the embedded length of the sheet piles or the concrete spur respectively is sufficient. Earth dams are more vulnerable to high flow velocities due to possible failure of the slope protection. With regard to combined groynes, it is an advantage of earth dams, that the transition between the impermeable and the permeable section of the groyne can be modelled more favourably to the flow thereby, limiting unfavourable flow conditions and as such the scour depth in this critical area.

The necessary materials and plant for construction of any of the considered types of impermeable groynes are available in Bangladesh. The work can be carried out by local contractors. The sheet piles and the materials for the slope and bed protection can be produced / procured in advance and stockpiled. The earth for the dam and the filling of the cofferdams are usually taken from borrow pits nearby, so that stock piling is not a problem worth mentioning. The concrete spurs are cast in-situ and can therefore not be stockpiled, except for the individual components like cement, aggregates or reinforcement steel.

The erosion of the flood plain during the monsoon flood of 1999 has caused the falling apron to displace partially (see Photo 8.2-6). No major damages to the concrete structure were observed.

One of the two groynes at Subogacha were destroyed completely by the flood in 2000. No details are available about the development and the reasons for the damages.

A similar groyne was built at Hasnapara in 1996. The 150m long concrete head was connected to the main embankment by an earth dam. Already during the first monsoon flood (after the completion of the groyne), the floodplain around the structure eroded to such an extent that the cc-block protection displaced and a large number of concrete piles became exposed to the river. In the following monsoon season, erosion of the flood plain progressed to such a large degree that finally more than 100 m of the concrete structure were entrained by the flood (Photo 8.2-7). To avoid the complete failure of the groyne, the toe of the remaining structure was protected by dumped concrete blocks.

Photo 8.2-7: Remnants of the groyne at Hasnapara (situation in December 1999)

<u>(e) Other Structures</u>

Under FAP 9B several types of revetment structures were studied and developed for the Meghna environment. The basic features are very similar to those of FAP 1, with the difference that the protective layers consist of boulders or rock instead of concrete blocks. However, no structure was built up to May 2001.

Further revetments were built or are presently under construction along the Ganges River and other major rivers in Bangladesh. However, no detailed information is available about these structures. Some of them are included in Table 8.2-1.

8.2.3 Assessment of Costs

A comprehensive cost assessment has to comprise the costs for land acquisition, construction, consultancy and other factors related to the construction of the structures.

However, the different basic concepts and execution methods as well as different financing sources and related types of construction contracts complicate the direct comparison of costs. Some items, such as the land acquisition and earth filling works, depend largely on the local situation and are not imperatively related to the type of structure. They can, therefore, not be included into a general cost comparison. This also holds for the consultancy costs which depend on the participation of foreign consultants or not.

All structures under the Jamuna Multi-Purpose Bridge (JMB) contract and FAP 1 (later River Bank Protection Project, RBPP) were designed and built mainly by international consultants and contractors. FAP 21 has a significantly different approach, with the Consultants being not only responsible for the design but also for the construction, monitoring and adaptation of the Test Structures. However, for the construction works, local contractors were employed.

Due to limited comparability of the structures and insufficient availability of data regarding the costs, a detailed comparison of the construction costs is not possible. The costs given in Table 8.2-1 therefore only represent an order of magnitude, which cannot be applied to future structures without further analysis. It must also be kept in mind that the structures were the first of their kind in terms of volume of work and execution time, so that it can be assumed that future structures of similar type might be cheaper.

The locally contributed costs are given in Taka (Tk) whereas the foreign financial contributions are given in US Dollar (US$) or Deutsche Mark (DM) respectively. For comparison purposes, an average exchange rate of 1 US$ = 40 Tk = 1.8 DM can be applied. As all structures were built within a five year span, no price escalation was applied.

8.2.4 Performance of Structures and Conclusions

Experience has shown that unprotected embankments do not provide sufficient protection against erosion at the major rivers in Bangladesh, due to missing or insufficient protection of the slopes and the riverbed around the structure.

Falling aprons of concrete blocks have performed well at Bahadurabad Test Site. However, they can not prevent slope slides under all conditions, as experienced at the FAP 1 "Hard Points", in particular at Sirajganj and Kalitola. Alternatively, geobags can be used. It must, however, be considered, that the permeability of the available geotextiles is too high to retain the local fine fill sand and silt on a long term basis, so that it is washed out partially. This also holds for sand-filled articulating geotextile mattresses.

Significant slope slides occurred during dredging of the 1V : 3.5H underwater slopes for the Jamuna Bridge guide bunds and Sirajganj Town Protection. A likely reason for this is the fast excavation of the slope which did not allow the soil to consolidate sufficiently. Therefore, underwater slopes should not be built steeper than about 1V : 6H.

Substantial scouring occurred downstream of the heads of impermeable groynes / groyne sections and the terminations of revetments protruding into the river. The scours were produced by return currents which themselves were induced by the separation of the flow in these areas. In order to minimize such negative return currents, special attention has to be given to the design of the heads and terminations of the structures.

A comparison of the implemented schemes shows that high investment costs do not necessarily prevent damages and maintenance measures. Cheaper structures, such as those developed under FAP 21, which make use of local techniques and construction methods, have the advantage that any damage can be repaired with less efforts and by making maximum use of local means. Any erosion protection structure must be monitored carefully and damages must be repaired immediately. According to the development of the river, preventive maintenance measures might become even more necessary in order to avoid future damages of the structures.

9 RECOMMENDATIONS FOR IMPLEMENTATION OF FUTURE EROSION PROTECTION MEASURES

9.1 GENERAL STRATEGY

A general strategy towards safe and cost effective measures against bank erosion along the major rivers of Bangladesh requires a large-scale framework, covering the entire projected area. Regional and sub-regional planning units responsible for monitoring certain smaller sections of a specific river will contribute substantially towards providing the morphological and hydraulic database.

A **S**trategic **M**aster **P**lan (SMP) is mandatory to organize and co-ordinate the temporal and spatial implementation process. Within this context the term implementation includes all relevant processes of planning, designing, constructing, monitoring and in some cases, of adapting a bank erosion protection structure. Suitable interfaces should ensure co-ordination and integration with the National Water Management Plan. The implementation process has to be performed in all directions taking into account new information from either side as a basis for the decision makers at national and regional planning level. The exchange of know-how and information is a central aspect in an optimised planning process, to allow for the correct choice and assessment up of different locations where bank protection is required and to co-ordinate immediate actions in cases of unexpected severe attack on banks near facilities of importance. The temporal and spatial implementation process of a bank protection structure is presented schematically in Fig. 9.1-1. The planning units as well as the temporal sequence may be used as a guideline. Due to the high mobility of the major rivers and the probability of occurrence of extreme hydraulic events, the process needs a continuous review and appropriate adaptation. Thus, a competent organization is a must and the core of a comprehensive and flexible arrangement of any key actions to be taken within the SMP.

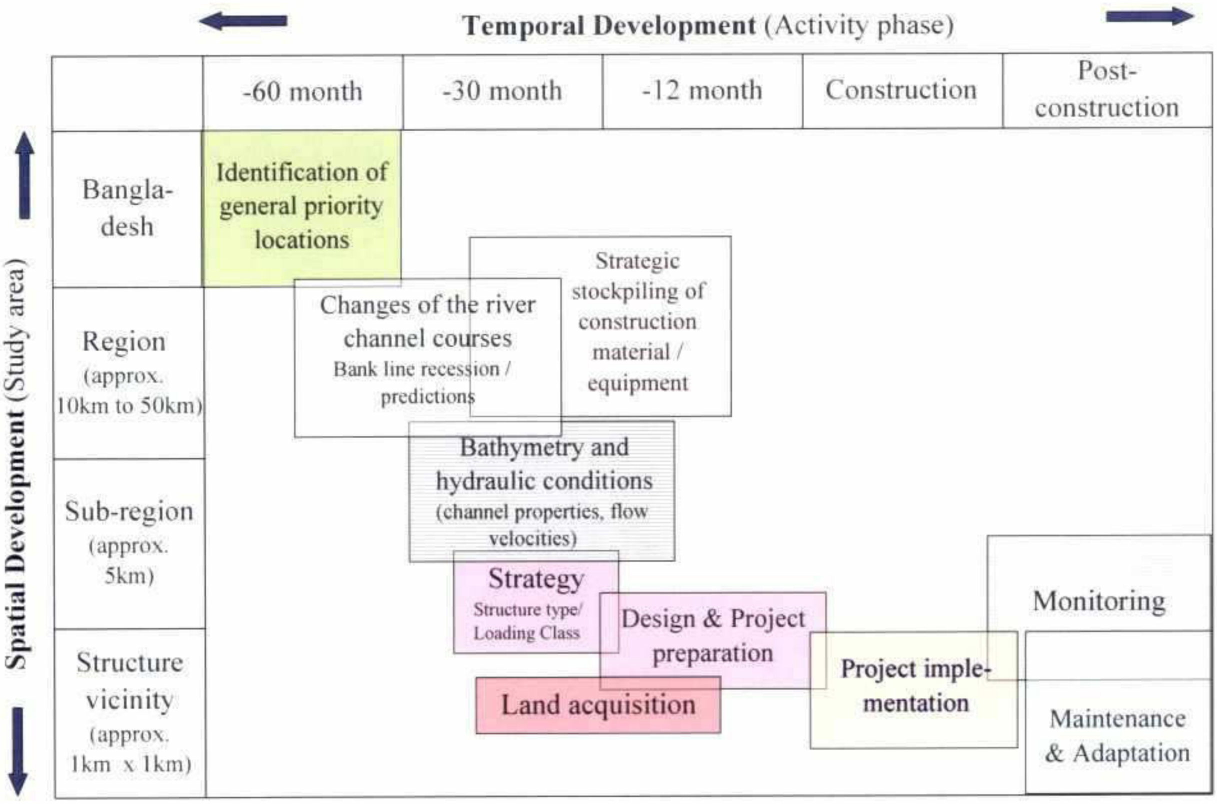

Fig. 9.1-1: Temporal and spatial development within the implementation process

The locations given priority can generally be simply defined by one major individual asset (e.g. a power plant, irrigation pump station, etc.) or by an accumulated number of assets (e.g. a village serving as a trade centre) located on the peripheries of a major river. These areas can be identified by GIS-mapping or by traditional/conventional methods combining topographical and socio-economical information, which have to be frequently updated in a priority atlas within the SMP.

Some aspects of the SMP operate in an overlapping mode between individual parallel or subsequent activities as shown in Fig. 9.1-1. For example, strategic stockpiling of construction material is necessary in advance for a very specific location and bank protection project but must also be available prior to construction start and for any required emergency repair or maintenance measure. Hence, the strategic planning and maintaining of countrywide material stockpiles will be an important and permanent task within the SMP. Moreover, it is a prerequisite for an effective SMP to provide sufficient financial resources for bank protection measures to avoid unacceptable delays within the implementation process.

If more than one of the pre-defined exceptionally valuable areas are endangered by bank erosion over a certain period of time, a priority ranking has to be carried out taking into consideration the anticipated morphological development as well as socio-economical and technical criteria. The general steps and the required information to be able to achieve a priority ranking for the most urgent measures is given in Fig. 9.1-2. The required basic information can be primarily provided by the regional/sub-regional Water Board Authorities, which should also become responsible for the preliminary concept of the envisaged structures (marked as blueish boxes). The Guidelines and Design Manual established within the FAP 21-Project present a suitable tool for this part of the planning process. However, due to the specific character of the **S**trategy for **I**dentification of **P**riority **P**rotection **S**ites (SIPPS), the key decisions should be taken at a superior level of the Water Board Authorities (light green boxes), also with due regard to the available, respectively the required funds.

Generally, the Multi-Criteria Analysis is a repetitive process over a designated amount of time and must be carried out on the basis of regularly updated data sets to achieve substantial reliability in the priority ranking. However, despite all of these precautions and prediction tools, it must always be considered that the dynamic behaviour of the major rivers of Bangladesh occasionally demands drastic changes of scheduled implementation plans.

9.2 COLLECTION OF PROJECT DATA

A comprehensive assessment of existing information and anticipated developments is the key input at every level and stage of the implementation process. The data collection process along with the scope and extent of the individual project stages are summarized in Fig. 9.2-1. The data collection and assessment program includes different temporal and spatial components to provide the essential information for the SMP. It also incorporates socio-economic surveys of the river peripheries which have to be updated frequently dependent on the area specific development regarding population density, economic viability, etc. Supplementary information could possibly be obtained from data sets established by the Bangladesh Bureau of Statistics and other governmental authorities or non-government organisations (NGO). All these inputs also form the basis for the evaluation and comparison of different locations within the general priority classification (SIPPS).

Simultaneously with the conceptual definition of a structural solution for an identified location, the channel properties and hydraulic conditions have to be further monitored to enhance the prediction of the likely development of the riverbank within subsequent years. This must be accompanied by a

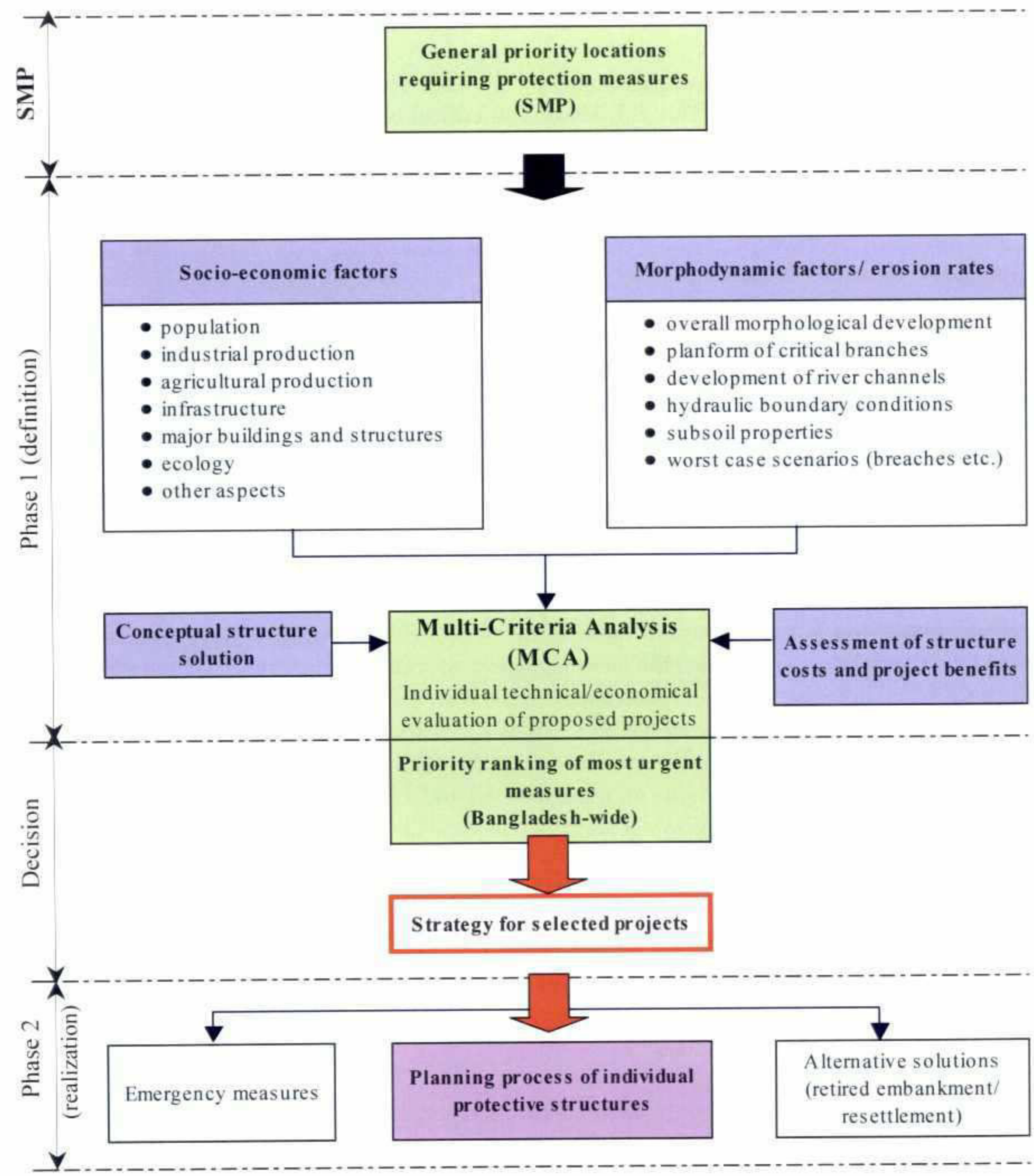

Fig. 9.1-2: Flow chart for determination of project priorities (SIPPS)

proposal on the structure type including the expected capital costs and assessed benefits as inputs for the MCA. Dependent on the category of structure chosen (refer to Subsection 7.2), the detailed structural design may require additional data and confirmation/correction of the anticipated morphological development in the specific area.

To extend the service life of the protection measures, monitoring of the river and inspections of the completed structures is essential. This will ensure that critical boundary conditions can be detected at an early stage, thus limiting not only the repair measures but also the related costs. These post-construction activities will also help to identify possible failure modes, to further improve the design of bank protection measures and construction techniques. Moreover, the comparison of estimated and actually measured hydraulic and morphological boundary conditions will improve basic design assumptions.

9.3 PLANNING PROCESS OF INDIVIDUAL MEASURES

A schematic sequence of activities for the definition, implementation and maintenance of individual bank protection structures is presented in Fig. 9.3-1. These activities should be carried out under the responsibility of District Water Board Authorities.

For Loading Class 1 minor activities combined with further observations are recommended. Standard Protection Structures (SPS), as defined in the Guidelines and Design Manual, are recommended for Loading Class 2 and 3 respectively. In the definition stage (Phase 1, Fig. 9.3-1), structural options should be studied. Decisions on the type of structure (i.e. revetment or groyne field) and the standardized structural components can be reached under consideration of the pros and cons regarding the site-specific boundary conditions as described in Chapter 7. After confirmation of the proposed structure by the superior Water Board Authorities subsequent to the MCA, further planning and preparation for the structure implementation has to be initiated. The details of individual planning processes and the related steps during the project realization, as shown in Fig. 9.3-1, are presented in the Guidelines and Design Manual.

Special structures as categorised by Loading Class 4, Subsection 7.2 (extraordinary hydraulic loads) require detailed investigations by national/ international experts (national expert panel, task force) and more in-depth studies, modelling and design procedures. Unquestionably, also such special structures must be included in the nation-wide Multi-Criteria-Analysis (see Fig. 9.1-2).

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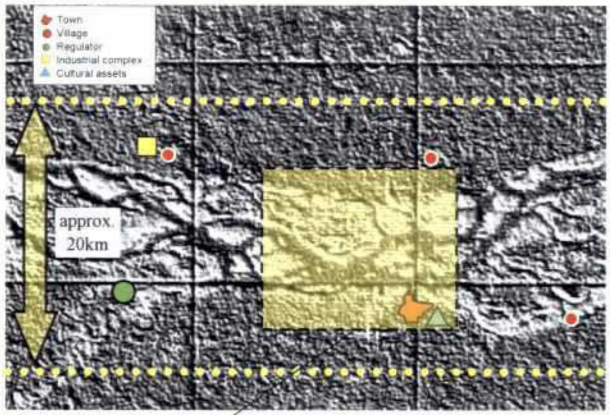
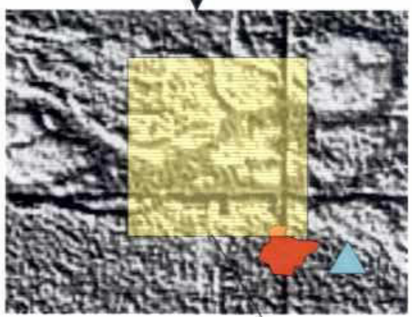
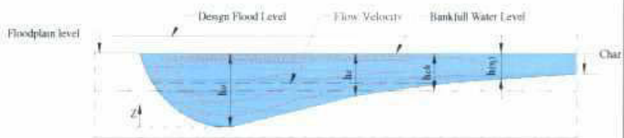
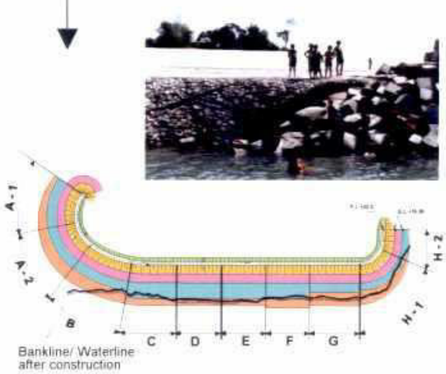
<table>
<tr><th></th><th>Description</th><th>Example</th></tr>
<tr><td>Definition of priority sites (SIPPS)</td><td>AREA:
Peripheries of the major rivers of Bangladesh (bank full river surface plus approx. 5km parallel to the river course)
SCOPE:
Definition of priority locations/ areas of high socio-economic value as general basic information for the Strategic Master Plan (SMP)
PARAMETERS:
• Population density;
• Industrial complexes;
• Trade centres;
• Agricultural production, and
• Cultural assets / national heritage.</td><td></td></tr>
<tr><td>Conceptual design</td><td>AREA:
At least 10km to 50km (mostly upstream of a potential area to be protected)
SCOPE:
Estimation of bankline and river channel development (annual reporting) in the vicinity of a general priority site which may possibly be endangered in following monsoon seasons
PARAMETERS:
• Frequent monitoring of banklines incl. major branches;
• Identification of main channel approach;
• Evaluation of erosion rates, and
• Estimation of hydraulic loading class.</td><td>approx.
10 km to 50 km along a river reach
</td></tr>
<tr><td>Design / Implementation</td><td>AREA:
At least 5km (in the vicinity of a potential structure location)
SCOPE:
Estimation of hydraulic and morphological boundary conditions. Start of detailed design process. Optimal structure location and layout
PARAMETERS:
• Bathymetry;
• Flow velocities;
• Scour location and depth;
• Loading class / structure type, and
• Area required for structure implementation.</td><td></td></tr>
<tr><td>Post-Construction</td><td>AREA:
Structure components and near field
SCOPE:
Inspection of structural conditions as basic information for maintenance, repair or adaptation measures. Investigation of causes and failure modes to further improve the design and technical knowledge
PARAMETERS:
• Structural behaviour
• Near field flow conditions</td><td></td></tr>
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Fig. 9.2-1: Collection of project data within future bank protection programs

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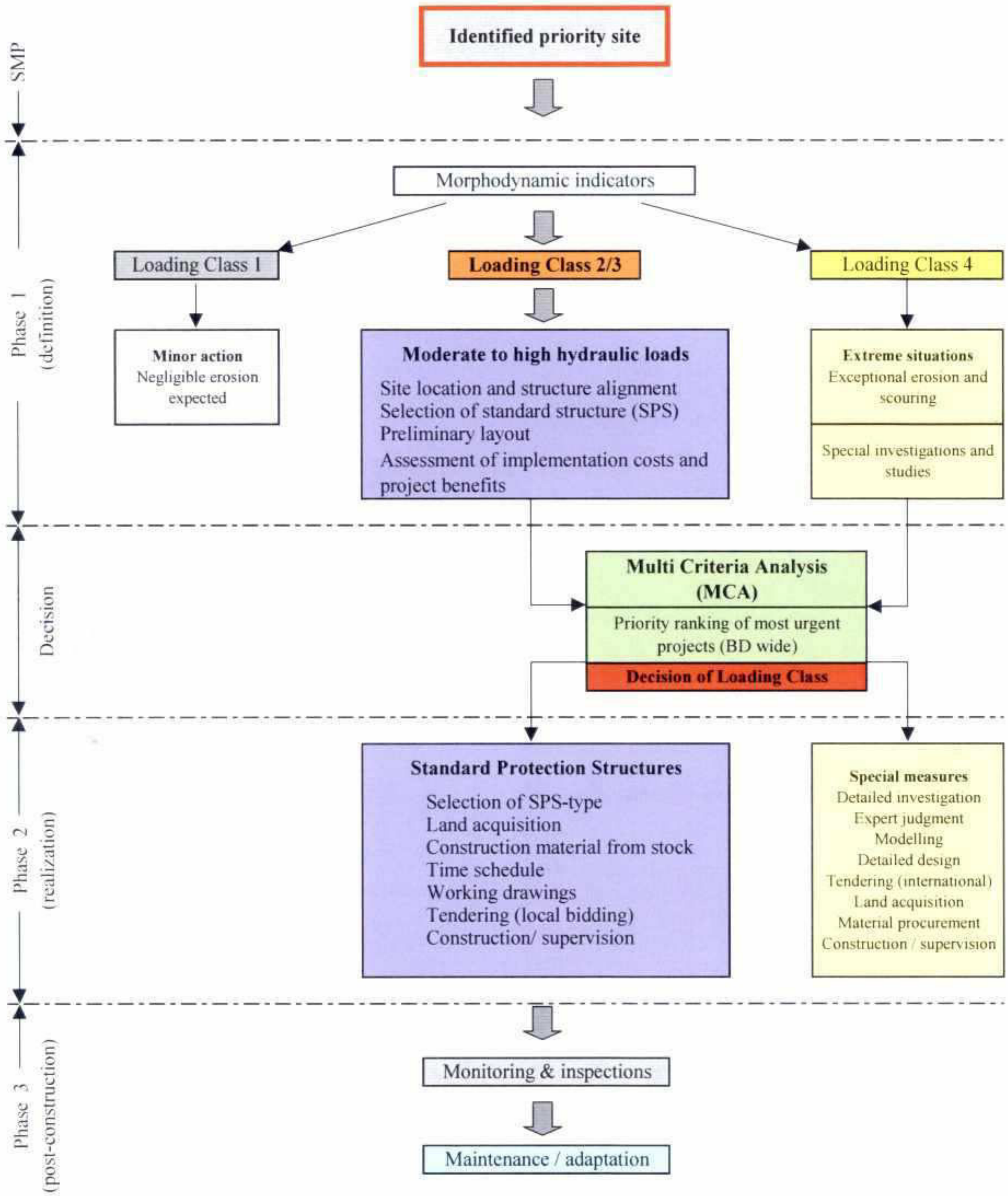

Fig. 9.3-1: Implementation process of individual bank protection measures

9.4 STANDARDIZED STRUCTURES AND SAFETY LEVELS

The utilization of standardized structures offers remarkable advantages when compared to individually planned measures against bank erosion as the planning process can be significantly accelerated. This is very important, especially after confirmation of the proposed project by the decision makers, since the time period between (positive) decision and implementation of a project is considered to be rather short. In addition, standardized structural elements and materials as well as construction equipment can be produced or procured sufficiently before construction start (delays in material supply were the reason for regular hindrance of work during the pilot project). The availability of actual dimensions and quantities of material needed therefore has to be confirmed during the planning process. The frequent application of standardized structural solutions against bank erosion will further increase the experience and know-how with these kinds of measures, provided that the structure behaviour and flow conditions are monitored and evaluated sufficiently after project completion.

The Standard Protection Structures (SPS) developed within the FAP 21-Project were designed at a lower safety level, e.g. as compared to so-called "hard points". To avoid uneconomical over-designing against maximum conceivable hydraulic loads, the expected hydraulic conditions of the forthcoming flood seasons at a specific location were used as design basis. This is justified when taking into account the difficulties involved in predicting the morphology of a river such as the Jamuna, particularly when the assumptions regarding the actual operating time of a structure as well as the prevailing river channel and flow properties and the structure-flow interference within it's life time are vague during the design process.

Due to the high mobility of braiding and anabranching rivers, the conventional approach would have to include the superposition of the most unfavourable conditions which would lead to an over-designed structure with the consequence of excessively high investment costs (uneconomic design, see Table 9.4-1). Experience gained from other structural measures show that even a massive structure might be inadequate to accommodate unexpected extraordinary loadings if it's position or the alignment is sub-optimal. Therefore, within FAP 21 an economically more viable approach was followed, which allows for a substantial reduction of the initial investment but may require slightly higher maintenance costs and, in some cases, an adaptation of the original layout. This design approach should certainly include some monsoon periods, where smaller hydraulic loads occur as compared to the design values. Contrary to this, in some cases extraordinary situations may occur, affecting the integrity of individual structure components but not the intended function in terms of bank protection (acceptable design). If the ratio of **structure service life** (for which the actual hydraulic loads are in the range of the design loads, green shading) versus **structure lifetime** approaches unity, the theoretical optimal design is achieved in economic terms. However, a certain probability of unexpectedly high loads (versus the initial design assumptions) within the structure service life can be tolerated. Therefore, structures that can be adapted and reinforced without difficulties after completion are to be favoured and are economically most feasible.

Fictitious structure life time (monsoon seasons)																														Design
1-10										11-20										21-30										
				M					M						M							M								uneconomic
				M						M			M			M			M			M				M				acceptable
		M			M			M			M			M			M			M			M			M			M	optimal
		M						M	R	A			M				M					M				M			M	adaptable

Actual loads « design loads; Actual loads ≈ design loads; Actual loads > design loads; M: maintenance R: repair A: adaptation

Table 9.4-1: Time and durability relationship over a fictitious structure life (schematic)

9.5 TYPICAL STRUCTURAL SOLUTIONS

Within the FAP 21 project, two typical structures were developed to protect river banks and adjacent land against erosion. Both types showed satisfactory and practicable results regarding the achieved hydraulic function and stability. Therefore, the Standard Protection Structures (SPS), presented in more detail in the Guidelines and the Design Manual, are designed on the fundamentals experienced during the project. In this section, they are briefly discussed in a rather conceptual frame. Generally, the recommended bank erosion protection measures followed a typical layout utilizing standardized designs for the individual structural components. Alternative materials or construction techniques described in the Design Manual allow for flexibility taking into account any local and temporal restrictions. The actual extent and alignment of the structure foot print and the dimensions of the structure elements have obviously to be designed in accordance with the existing boundary conditions.

A typical layout for a groyne field consists of at least three successive groynes, covering about 1 km of river bank. Naturally, the length - distance relationship must be chosen individually for the specific situation. To prevent extensively large eddy formation behind an individual groyne structure due to flow separation at the head of impermeable or partially impermeable (towards the root) groynes, the use of completely permeable structures is strongly recommended. A typical groyne is constructed of single tubular impact driven steel or pre-stressed concrete piles or of cast in-situ concrete piles, respectively. The distance between the individual piles should preferably be designed variable from the root to the head of the groyne (Fig. 3.6-1) or should increase gradually, with a porosity ranging between 40/50 % and 80 % (Fig. 9.5-1).

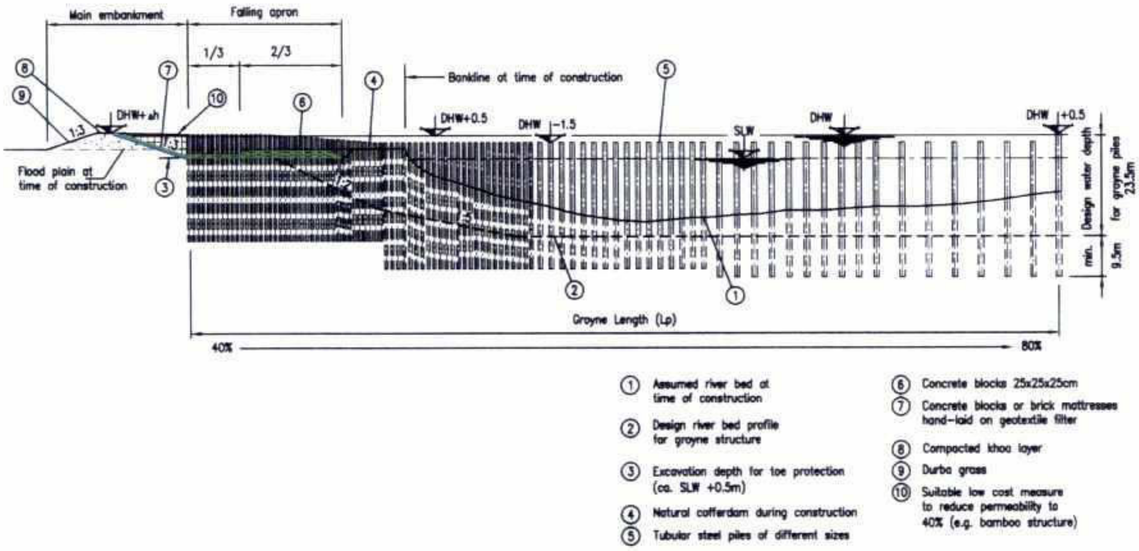

Fig. 9.5-1: Typical permeable groyne (cross-section)

The acceleration of the flow velocity due to the water level gradient at the groyne axis, followed by generation of local vortices, requires some scour protection in the vicinity of the embankment, e.g. a scour blanket made from concrete blocks (Fig. 9.5-2). The outer slope of the main embankment to which the individual groynes are connected, has to be protected by an armour layer capable of resisting the residual near bank flow velocities.

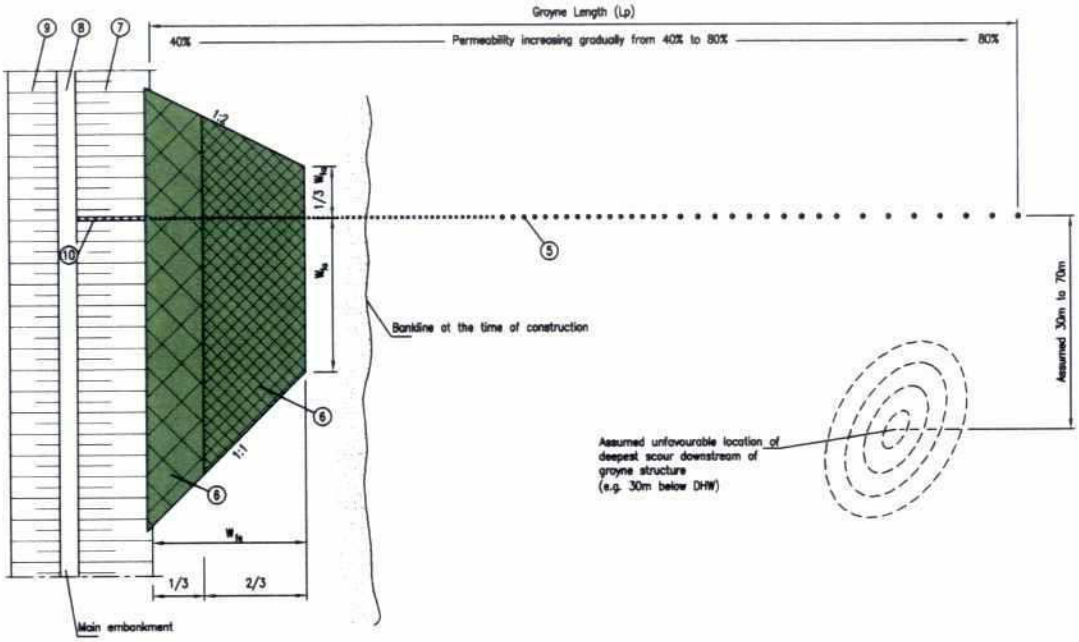

Fig. 9.5-2: Typical permeable groyne (plan view)

A typical revetment structure is formed by a straight part supplemented by upstream and downstream terminations (Fig. 9.5-3), all of which are protected by falling aprons of concrete blocks to stabilize a potential scour hole. Along the straight part of such structures the falling aprons are extended and reinforced by interconnected concrete blocks or stone filled wire mesh mattresses (launching apron). These have shown promising performance during the pilot phase of the Project. Scour protections may be placed directly on the dry floodplain in front of the main embankment, but (besides other advantages) due to the fact that earth material is needed for the dyke construction anyway, it is recommended to excavate the required material from this area, as long as a sufficient freeboard during the construction time is affirmed (Fig. 9.5-4). To prevent subsoil erosion underneath the armour layer, it is important to place a geotextile with adequate properties prior to the cover layer (stretch between structure crest and falling apron). The revetment of the main embankment should preferably be armoured by a single layer of concrete blocks, whereas the crest should be prepared with a compacted khoa layer. For the construction of revetments a large number of local workers can be employed, which contributes to the positive socio-economic impact of this labour intensive measure.

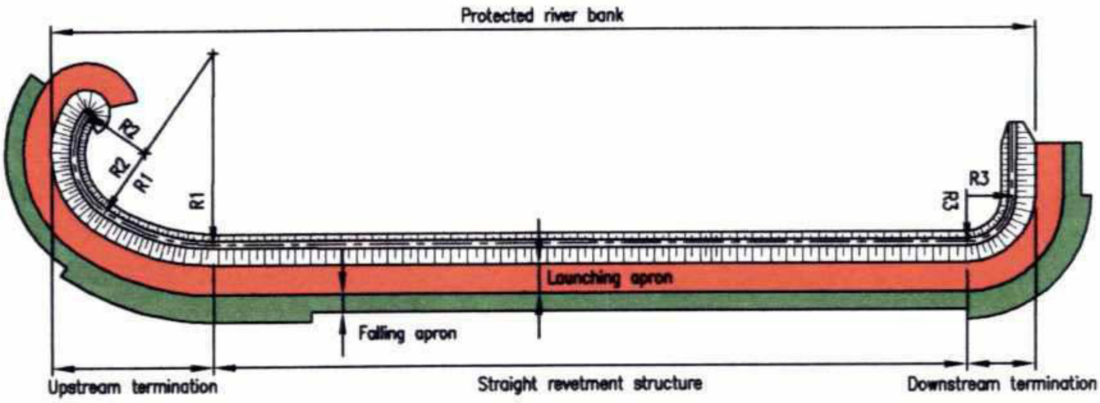

Fig. 9.5-3: Typical revetment structure (plan view)

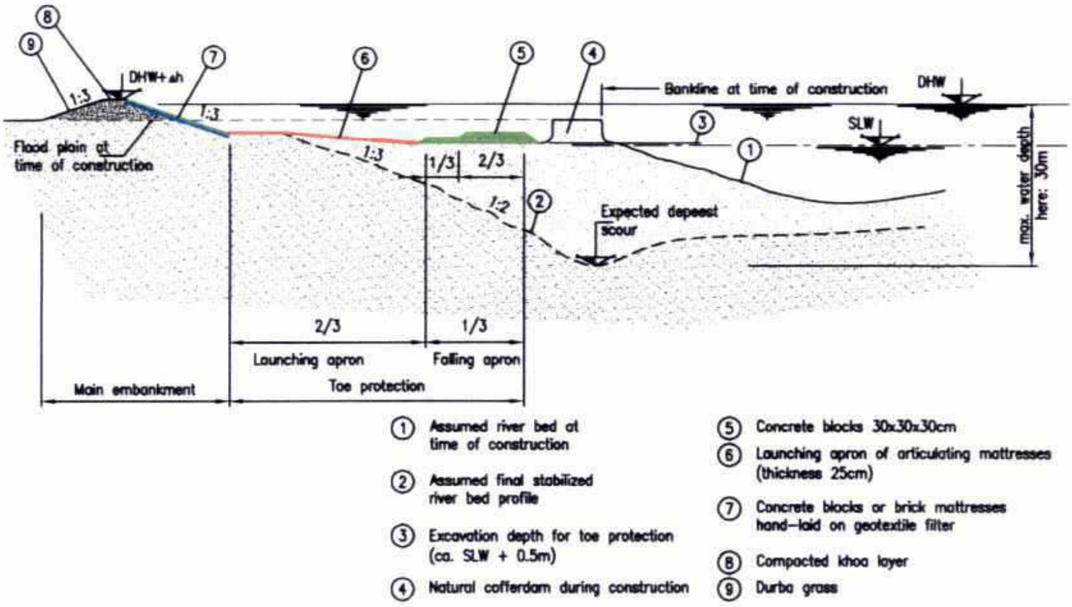

Fig. 9.5-4: Typical revetment structure (cross-section)

9.6 ECONOMIC EVALUATION AND COMPARISON OF PROPOSED STRUCTURE TYPES

In this section, a brief discussion on the economic viability of the two proposed typical structure types is given. The cost calculations were made on the basis of the overall layout and dimensions of structural components, assuming hydraulic and morphologic boundary conditions as described for Loading Class 3. The hypothetical structure length was chosen to b = 1000 m, which was also taken as the protected stretch of the bankline. The implementation costs take into account the capital investment (land acquisition/compensation, construction and general costs) as well as the recurrent costs used for future monitoring and maintenance, as described in Chapter 6. Consulting costs are not included because it is assumed that for SPS no extensive design studies are required. Other activities within the general planning process can be carried out by BWDB personnel.

The construction costs of a groyne field are considerably lower (75 %) as compared to the construction costs of a revetment (Table 9.6-1). In this context, it should be kept in mind, that the up- and downstream terminations (which are more or less fixed due to the minimum radius defined) contribute considerably to the revetment costs, i.e. the costs per metre of protected bankline are dependent on the structure length.

To allow for a comparison between a groyne field and a revetment structure, the accumulated assets and the industrial/agricultural production in the fictitious protected area were taken similar to the estimated values for the Bahadurabad area (Kulkandi). It has to be emphasized, that this approach is quite conventional because the socio-economic indicators of this region are rather low (land value: 200,000 Tk/ha, annual production: 50,000 Tk/ha).

Investment Costs Standard Groyne Field (sum total) (financial current prices)		
Item	**Taka**	**Taka**
Land acquisition/ compensation		
(approx. 6 ha)		**4,800,000**
Construction (year 0 - 1)		
Embankment (earthworks)	8,040,000	
Embankment (revetment)	32,597,500	
Pile-Sections	114,016,500	
Toe protection	27,520,000	
General[1]	27,326,000	
Total (investment costs)		**209,500,000**
Recurrent costs		
Monitoring	1,820,000	
Maintenance/ repair	2,404,700	
Total (recurrent costs)		**4,224,700**

[1] site installation, special equipment, etc. (15%)

Investment Costs Standard Revetment (sum total) (financial current prices)		
Item	**Taka**	**Taka**
Land acquisition/ compensation		
(approx. 10 ha)		**8,000,000**
Construction (year 0 - 1)		
Earthworks	19,050,000	
Revetment	41,200,000	
Launching apron	82,850,000	
Falling apron	118,900,000	
General[1]	18,340,000	
Total (investment costs)		**280,340,000**
Recurrent costs		
Monitoring	1,820,000	
Maintenance/ repair	4,962,000	
Total (recurrent costs)		**6,782,000**

[1] site installation, special equipment, etc. (7%)

Table 9.6-1: Estimated investment and recurrent costs for 1000m protected bankline, based on standardized design (Loading Class 3)

The annual erosion rate at the fictitious site location was varied between 20 m and 225 m, to calculate the respective Net Present Value of the project over the projected structure life time of 30 years (Fig. 9.6-1). The financial break-even point of the protection measures is achieved at an annual erosion of approx. 90 m (groyne field) and approx. 110 m (revetment), respectively. Assuming an erosion rate of 150 m, the accumulated NPV after 30 years totals between 150 and 200 million Tk, depending on the structure type. These promising results accentuate the economic design and feasibility of standard protection structures.

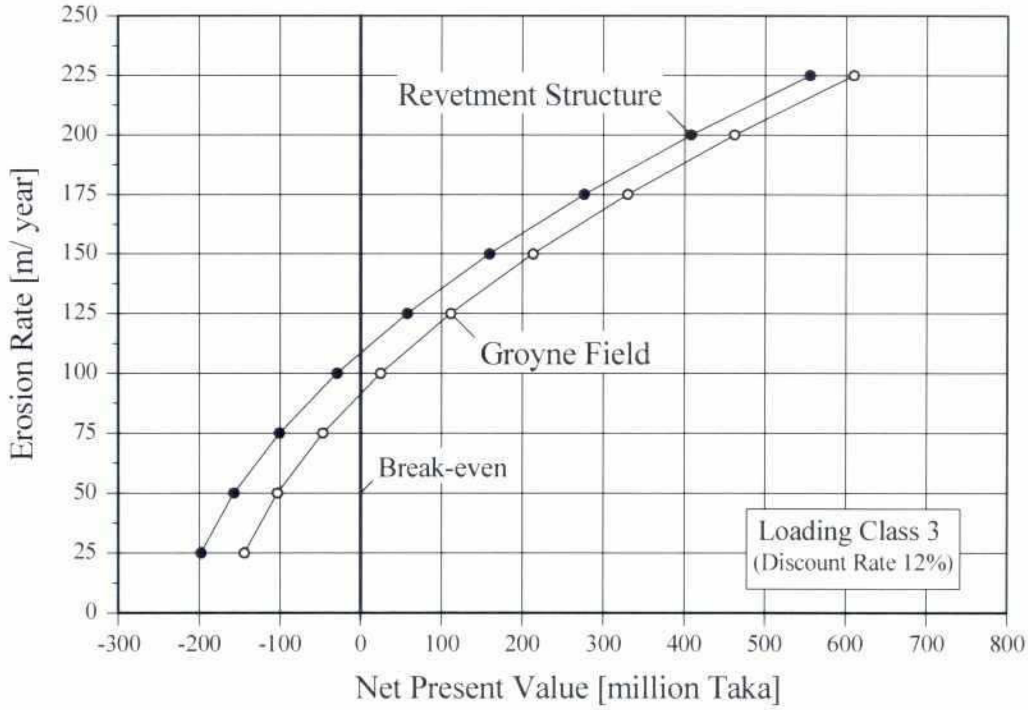

Fig. 9.6-1: Net Present Value under assumption of different annual erosion rates for Standard Protection Structures (Groyne Field and Revetment)

Furthermore, it can be stated, that for a high ranking general priority site, defined by an exceptional prosperous area in socio-economic terms, the basic parameters for the example calculation have to be adjusted. In case of a well developed township, the associated socio-economic factors would result in much higher land prices and a higher productivity rate. Multiplying the basic values by a factor of 2.5 (land value: 500,000 Tk/ha, annual production: 125,000 Tk/ha) seems to be a fair estimate to describe a more realistic scenario for future bankline protection measures. Even if the structure service life (discussed in Section 9.4) is restricted to ten years - i.e. significantly reducing the net return of the project - the achieved EIRR is in the range of the assumed discount rate (12 %) if the annual erosion rate exceeds 135 m (groyne field) or 165 m (revetment structure). The calculations of EIRR and NPV, following the described scenarios for SPS, are given in Attachment 2.

10 CONCLUDING REMARKS

10.1 PRIMARY FOCUS AND ACHIEVEMENTS

The FAP 21 – Bank Protection Pilot Project – was aimed to develop, test and improve safe and economic structural measures to cope with substantial losses and intangibles, resulting from ongoing severe bank erosion along the major rivers of Bangladesh. The major aim was to investigate measures which allow for feasible future solutions in terms of material availability, construction techniques and restricted construction times.

During the early stage of the implementation phase of the Project (1993-96) two prototype structures, a groyne field and a revetment structure, were developed. The realization of the structures was achieved at comparably low cost and the utilized funds for maintenance and adaptation remained much below initially planned budgets. In agreement with the Donors, this allowed the set-up of a third prototype structure, for which an optimized revetment design was chosen.

After construction, the structures were subjected to the hydraulic loads of the Jamuna river during the successive monsoon seasons. Throughout the test phase of the Project, the structural performance and the morphological changes were monitored and analyzed. Where necessary, adaptation and maintenance measures at certain structural components were carried out.

Throughout the project monitoring phase (1995 to 2001), all structures have fully protected the adjacent hinterland. Furthermore, the substantially reduced risk of erosion induced losses contributed particularly to the very prosperous development in the respective regions.

The prototype structures were designed at a lower safety level, to define critical boundary conditions regarding dimensions of structure elements and prevailing hydraulic loads. Nevertheless, apart from some corrective measures for the groyne field, the prototype structures showed very good performance and the negative morphological side effects remained tolerable.

In general, the tested and improved prototypes as well as the applied construction methods have proven to be fully reproducible in Bangladesh. At present, only some materials (large diameter steel piles, geotextile filter materials and special wiremesh boxes) may have to be imported, but will be available in Bangladesh as soon as the respective demand attracts the local business community.

Standard Protection Structures

At this stage it appears viable to recommend typical structural solutions for certain future erosion countermeasures at the major rivers of Bangladesh. The application of these measures should be restricted to moderate/high flow attack and scouring conditions. At locations where excessive hydraulic loads are expected, the boundary conditions and structural solutions must be investigated systematically and in detail, involving to a large extent expert judgement and model studies.

As a basis for future standardized structures, the following key experiences were determined during the structure implementation:

- Revetment structures should be fully constructed in dry condition because of the substantial advantages in terms of sliding stability, construction time and the smaller requirements regarding appropriate equipment.

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- In general, fully permeable groyne structures should be preferred to combined groynes. Any structure configuration that would induce excessive vortices, return currents and large eddy formation in the groyne fields must be avoided.

- Falling aprons, which have demonstrated high efficiency to protect and stabilize an eroding bank or scour hole, should be preferably built with suitably sized concrete blocks.

Considering the range of loading classes as defined in Subsection 9.3, it is assumed that the FAP 21 Guidelines and the Design Manual will efficiently assist the river engineers of Bangladesh throughout the planning and implementation of standardized structure types.

Strategy

The introduction of standardized structures for future bank protection measures in Bangladesh bears fundamental advantages. Besides a considerable acceleration of the planning process, which is crucial in case of highly dynamic rivers and restricted construction windows, the actual construction time as well as the costs for material and implementation can be reduced significantly.

The high mobility of the river course substantiates the basic idea of designing structures at a reduced safety level, allowing for partial damages at certain structure components, but decreasing the capital investment. This approach is inseparably connected to the acceptance of certain repair and adaptation works during the structure service life. The economic viability of future measures will be decisive in the planning process, to decide the sequence of proposed activities at general priority sites.

The following key aspects and activities regarding the recommended future planning strategy accrued during the FAP 21 project phases:-

- Strategic Master Plan (SMP): The presently operational planning procedure should be adjusted and refined to include standardized structural solutions and to optimize the planning process. A critical point, which must be solved, is the process of land acquisition. This aspect is strongly interconnected to the support and participation of the local population. In this context, it must be emphasized that trained and educated villagers are capable of taking responsibility for the security of bank protection structures, provided they are given the necessary support and authority.

- Allocation of general priority sites (SIPPS): The definition of areas in the reach of major rivers, representing certain socio-economic indicators, is essential, to allow for an economic evaluation and a priority ranking of different measures.

- Strategic stockpiling: The utilization of standardized structural elements facilitates the storage of basic construction materials in strategic yards, which helps to prevent from delays during the construction process and to allow for immediate emergency measures.

- Monitoring: The inspection of structures after completion and the monitoring of the river morphology/hydraulics will be the basic source of new expertise to further optimize the structural design. A national expert panel should gather and analyze the available and newly collected data material and co-ordinate the necessary actions.

In this context the steps initiated by the BWDB through the FAP 21 monitoring activities are very encouraging and will be of fundamental importance for the success of future actions.

10.2 RECOMMENDATIONS FOR THE FUTURE

In order to take full advantage of the findings and experiences within the FAP programme and similar projects on river research carried out in Bangladesh, it is important to form a specific "Task Force" within the existing authorities (WARPO) to ensure the initiation and co-ordination of the required steps included in the Strategic Master Plan (SMP), the Strategy for Identification of Priority Sites (SIPPS) and the Multi-Criteria Analysis (MCA). Such "Task Force" should include the competence of morphologists and river engineers of Bangladesh through collaboration with existing institutions (EGIS, SWMC, etc.), to gather experience and data material from all available resources. In some cases, expatriate services might be advantageous to support institutional, strategic, financial and technical issues at certain planning levels to accelerate the implementation process.

The main findings presented in this report regarding bank protection measures are based on the experience gained during the FAP 21 field investigations. Considering the very complex processes involved, field investigations allow for more reliable results in terms of structure stability and morphological changes as compared to physical or numerical models. Nevertheless, due to the limited variability of prevailing hydraulic and morphological conditions in field studies, a frequent updating of the Guidelines and the Design Manual by introducing future project experience is mandatory, allowing to further optimize the proposed structure types.

Finally, it has to be emphasized, that the Guidelines and the Design Manual shall assist the engineers to efficiently plan and thus to accelerate the implementation of future bank protection measures. However, it should be kept in mind, that a standardized planning process – as described in the Guidelines and the Design Manual – need adjustment, in some cases, by local experience and expert/engineer judgement, especially for the Jamuna which is locally called "the crazy river".

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Attachment 1

Temporal Development of Capital Investment and Economic Costs for the Pilot Structures

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ATTACHMENT-1

LIST OF TABLES

Investment Costs: Kamarjani Groynes (current prices, incl. expatriate and local consultants)								
Investment costs	1993	1994	1995	1996	1997	1998	1999	Total
Land								
Land acquisition	3,573,529							**3,573,529**
Land lease (temp.)	2,500,000							**2,500,000**
Construction								
Constr contract		37,787,645	91,769,393	3,952,150				**133,509,188**
Material		79,605,735	6,625,523	88,923				**86,320,181**
Equipment (JTWC) [1]		45,400,650	14,778,904					**60,179,554**
Main Embankment [2]		5,170,796	5,170,796					**10,341,592**
Adaptation/Extension								
Adapt./Ext. Contract			1,396,774	75,005,130	7,739,550	7,970,621	3,114,031	**95,226,106**
Material				34,397,804				**34,397,804**
Equipment			36,979	3,404,015				**3,440,994**
Monitor./Mainten.								
Monitoring		8,906,902	4,116,038	2,907,581	5,285,671	2,247,474	607,364	**24,071,030**
Maintenance [3]			17,183,934	17,774,242	1,019,181	1,049,610	410,071	**37,437,038**
Other								
Port charge/demur./ ins.		7,751,297	2,104,679	2,014,491				**11,870,467**
Other		4,423,938						**4,423,938**
Consultant	57,812,412	22,880,413	14,576,541	16,808,829	1,691,720	1,357,253	497,656	**115,624,824**
Total	**63,885,941**	**211,927,376**	**140,759,561**	**156,353,165**	**15,736,122**	**12,624,958**	**4,629,122**	**605,916,245**
Realization [%]	10.5%	35.0%	23.2%	25.8%	2.6%	2.1%	0.8%	
Cumulative [%]	10.5%	45.5%	68.8%	94.6%	97.2%	99.2%	100.0%	100.0%

[1] Special equipm./ expatr.operators [2] borne by BWDB

[3] labour, material (incl. future reserve), etc

Table 1: Total investment costs (current prices) for the Kamarjani Pilot Structure

Economic Costs: Kamarjani Groynes (constant prices, incl. expatriate and local consultants)								
Investment costs	1993	1994	1995	1996	1997	1998	1999	Total
Land								
Land acquisition	5,110,146							**5,110,146**
Land lease (temp.)	3,575,000							**3,575,000**
Construction								
Constr contract		39,571,222	89,741,289	3,621,355				**132,933,866**
Material		83,363,126	6,479,099	81,480				**89,923,705**
Equipment (JTWC) [1]		47,543,561	14,452,290					**61,995,851**
Main Embankment [2]		5,414,858	5,056,521					**10,471,379**
Adaptation/Extension								
Adapt./Ext. Contract			1,365,905	68,727,201	6,912,966	6,812,490	2,517,694	**86,336,256**
Material				31,518,708				**31,518,708**
Equipment			36,162	3,119,099				**3,155,261**
Monitor./Mainten.								
Monitoring		9,327,308	4,025,073	2,664,216	4,721,161	1,920,916	491,054	**23,149,729**
Maintenance [3]			16,804,169	16,286,538	910,333	897,102	331,542	**35,229,684**
Other								
Port charge/demur./ ins.		8,117,158	2,058,166	1,845,878				**12,021,202**
Other		4,632,748						**4,632,748**
Consultant	71,924,422	27,072,104	16,105,621	17,402,181	1,707,284	1,310,699	454,609	**135,976,919**
Total	**80,609,568**	**225,042,084**	**139,124,296**	**145,266,655**	**14,251,744**	**10,941,207**	**3,794,899**	**619,030,453**
Realization [%]	13.0%	36.4%	22.5%	23.5%	2.3%	1.8%	0.6%	
Cumulative [%]	13.0%	49.4%	71.9%	95.3%	97.6%	99.4%	100.0%	100.0%

[1] Special equipm./ expatr.operators [2] borne by BWDB

[3] labour, material (incl. future reserve), etc.

Table 2: Total economic costs (at constant 2000 prices) for the Kamarjani Pilot Structure

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Investment Costs: Kamarjani Groynes (current prices, adjusted costs for pilot structure)								
Investment costs	1993	1994	1995	1996	1997	1998	1999	Total
Land								
Land acquisition	853,556							853,556
Land lease (temp.)	2,500,000							2,500,000
Construction								
Constr. contract		41,566,410	100,946,332	4,347,365				146,860,107
Material		87,566,309	7,288,075	97,815				94,952,199
Equipment (JTWC) [1]		22,700,325	7,389,452					30,089,777
Adaptation/Extension								
Adapt./Ext. Contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring								
Maintenance [2]								
Other								
Port charge/demur./ins.		3,875,649	1,052,340	1,007,246				5,935,234
Other								
Consultant	29,388,752							29,388,752
Total	**32,742,308**	**155,708,692**	**116,676,199**	**5,452,426**	**0**	**0**	**0**	**310,579,624**
Realization [%]	10.5%	50.1%	37.6%	1.8%	0.0%	0.0%	0.0%	
Cumulative [%]	10.5%	60.7%	98.2%	100.0%	100.0%	100.0%	100.0%	100.0%

[1] Special equipm./ expatr. operators

[2] labour, material (incl. future reserve), etc.

Table 3: Adjusted investment costs (current prices) for the Kamarjani Pilot Structure

Economic Costs: Kamarjani Groynes (constant prices, adjusted costs for pilot structure)								
Investment costs	1993	1994	1995	1996	1997	1998	1999	Total
Land								
Land acquisition	1,220,585							1,220,585
Land lease (temp.)	3,575,000							3,575,000
Construction								
Constr. contract		43,528,344	98,715,418	3,983,491				146,227,253
Material		91,699,438	7,127,009	89,628				98,916,075
Equipment (JTWC) [1]		23,771,780	7,226,145					30,997,925
Adaptation/Extension								
Adapt./Ext. Contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring								
Maintenance [2]								
Other								
Port charge/demur./ins		4,058,579	1,029,083	922,939				6,010,601
Other								
Consultant	36,562,546							36,562,546
Total	**41,358,131**	**163,058,142**	**114,097,655**	**4,996,058**	**0**	**0**	**0**	**323,509,986**
Realization [%]	12.8%	50.4%	35.3%	1.5%	0.0%	0.0%	0.0%	
Cumulative [%]	12.8%	63.2%	98.5%	100.0%	100.0%	100.0%	100.0%	100.0%

[1] Special equipm./ expatr. operators

[2] labour, material (incl. future reserve), etc.

Table 4: Adjusted economic costs (at constant 2000 prices) for the Kamarjani Pilot Structure

Investment Costs: Bahadurabad Revetments (current prices, incl. expatriate and local consultants)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land[*)]								
Land acquisition		6,403,460	15,354,275					**21,757,735**
Land lease (temp.)		3,700,000						**3,700,000**
Construction								
Constr. Contract		46,896,768	104,638,093	112,970,391	7,015,183			**271,520,435**
Material		52,962,542	14,483,716	18,330,236				**85,776,494**
Equipment		10,759,002	8,655,968	5,292,919				**24,707,889**
Adaptation/Extension								
Adapt./ ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring	81,142	5,658,581	3,793,022	1,936,858	4,316,664	3,198,545		**18,984,812**
Maintenance					6,591,827	2,445,787	113,714	**9,151,328**
Other								
port charge/demur./ ins.		6,064,533	2,335,790					**8,400,323**
Other							111,338	**111,338**
Consultant	53,213,697	19,874,159	15,970,920	14,822,760	1,917,834	603,944	24,081	**106,427,394**
Total	**53,294,839**	**152,319,045**	**165,231,784**	**153,353,164**	**19,841,508**	**6,248,276**	**249,133**	**550,537,748**
Realization [%]	9.7%	27.7%	30.0%	27.9%	3.6%	1.1%	0.0%	
Cumulative [%]	9.7%	37.3%	67.4%	95.2%	98.8%	100.0%	100.0%	100.0%

*) Phase 1 and 2

Table 5 Total investment costs (current prices) for the Bahadurabad Pilot Structure

Economic Costs: Bahadurabad Revetments (constant prices, incl. expatriate and local consultants)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land[*)]								
Land acquisition		8,132,394	18,271,587					**26,403,981**
Land lease (temp.)		4,699,000						**4,699,000**
Construction								
Constr. Contract		44,073,583	92,144,305	96,973,784	5,762,271			**238,953,942**
Material		49,774,197	12,754,360	15,734,675				**78,263,232**
Equipment		10,111,310	7,622,445	4,543,442				**22,277,197**
Adaptation/Extension								
Adapt./ ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring	81,661	5,317,934	3,340,135	1,662,599	3,545,708	2,485,269		**16,433,307**
Maintenance					5,414,527	1,900,376	84,148	**7,399,052**
Other								
port charge/demur./ ins.		5,699,448	2,056,897					**7,756,345**
Other							82,390	**82,390**
Consultant	62,962,446	21,958,959	16,534,693	14,959,129	1,852,052	551,703	20,950	**118,839,932**
Total	**63,044,108**	**149,766,825**	**152,724,422**	**133,873,628**	**16,574,558**	**4,937,349**	**187,489**	**521,108,378**
Realization [%]	12.1%	28.7%	29.3%	25.7%	3.2%	0.9%	0.0%	
Cumulative [%]	12.1%	40.8%	70.1%	95.8%	99.0%	100.0%	100.0%	100.0%

*) Phase 1 and 2

Table 6: Total economic costs (at constant 2000 prices) for the Bahadurabad Pilot Structure

Investment Costs: Bahadurabad Revetments (current prices, adjusted costs for pilot structure)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land								
Land acquisition		6,403,460						**6,403,460**
Land lease (temp.)		3,700,000						**3,700,000**
Construction								
Constr. Contract		25,963,757	115,101,902	124,267,430	7,716,701			**273,049,790**
Material		58,258,796	15,932,088	20,163,260				**94,354,143**
Equipment		5,379,501	4,327,984	2,646,460				**12,353,945**
Adaptation/Extension								
Adapt./ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring								
Maintenance								
Other								
port charge/demur./ins.		3,032,267	1,167,895					**4,200,162**
Other								
Consultant	36,969,876							**36,969,876**
Total	**36,969,876**	**102,737,780**	**136,529,869**	**147,077,149**	**7,716,701**			**431,031,376**
Realization [%]	8.6%	23.8%	31.7%	34.1%	1.8%	0.0%	0.0%	
Cumulative [%]	8.6%	32.4%	64.1%	98.2%	100.0%	100.0%	100.0%	100.0%

Table 7: Adjusted investment costs (current prices) for the Bahadurabad Pilot Structure

Economic Costs: Bahadurabad Revetments (constant prices, adjusted costs for pilot structure)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land								
Land acquisition		8,132,394						**8,132,394**
Land lease (temp.)		4,699,000						**4,699,000**
Construction								
Constr. Contract		24,400,738	101,358,735	106,671,162	6,338,498			**238,769,134**
Material		54,751,617	14,029,796	17,308,142				**86,089,555**
Equipment		5,055,655	3,811,223	2,271,721				**11,138,599**
Adaptation/Extension								
Adapt./ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring								
Maintenance								
Other								
port charge/demur./ins.		2,849,724	1,028,448					**3,878,172**
Other								
Consultant	43,742,757							**43,742,757**
Total	**43,742,757**	**99,889,128**	**120,228,203**	**126,251,025**	**6,338,498**			**396,449,611**
Realization [%]	11.0%	25.2%	30.3%	31.8%	1.6%	0.0%	0.0%	
Cumulative [%]	11.0%	36.2%	66.6%	98.4%	100.0%	100.0%	100.0%	100.0%

Table 8: Adjusted economic costs (at constant 2000 prices) for the Bahadurabad Pilot Structure

Investment Costs: Ghutail Revetments (current prices, incl. expatriate and local consultants)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land								
Land acquisition						7,647,083		**7,647,083**
Land lease (temp.)							595,430	**595,430**
Construction								
Constr. Contract						55,520,008	93,485,654	**149,005,662**
Material						7,129,757	2,766,744	**9,896,501**
Equipment								
Adaptation/Extension								
Adapt./ ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring							1,394,683	**1,394,683**
Maintenance								
Other								
Consultant					20,365,736	8,457,508	11,908,228	**40,731,472**
Other							7,025,375	**7,025,375**
Total	0	0	0	0	**20,365,736**	**78,754,355**	**117,176,115**	**216,296,206**
Realization [%]	0.0%	0.0%	0.0%	0.0%	9.4%	36.4%	54.2%	
Cumulative [%]	0.0%	0.0%	0.0%	0.0%	9.4%	45.8%	100.0%	100.0%

Table 9: Total investment costs (current prices) for the Ghutail Pilot Structure

Economic Costs: Ghutail Revetments (constant prices, incl. expatriate and local consultants)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land								
Land acquisition						8,029,437		**8,029,437**
Land lease (temp.)							595,430	**595,430**
Construction								
Constr. Contract						41,390,166	66,374,814	**107,764,980**
Material						5,315,233	1,964,389	**7,279,622**
Equipment								
Adaptation/Extension								
Adapt./ ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring							990,225	**990,225**
Maintenance								
Other								
Consultant					19,667,191	7,725,933	10,360,158	**37,753,283**
Other							4,988,016	**4,988,016**
Total					**19,667,191**	**62,460,770**	**85,273,033**	**167,400,994**
Realization [%]	0.0%	0.0%	0.0%	0.0%	11.7%	37.3%	50.9%	
Cumulative [%]	0.0%	0.0%	0.0%	0.0%	11.7%	49.1%	100.0%	100.0%

Table 10: Total economic costs (at constant 2000 prices) for the Ghutail Pilot Structure

Investment Costs: Ghutail Revetments (current prices, adjusted costs for pilot structure)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land								
Land acquisition						7,647,083		**7,647,083**
Land lease (temp.)							595,430	**595,430**
Construction								
Constr. Contract						55,520,008	93,485,654	**149,005,662**
Material						7,129,757	2,766,744	**9,896,501**
Equipment								
Adaptation/Extension								
Adapt./ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring							1,394,683	**1,394,683**
Maintenance								
Other								
Consultant					16,932,919			**16,932,919**
Other								
Total	**0**	**0**	**0**	**0**	**16,932,919**	**70,296,848**	**98,242,512**	**185,472,278**
Realization [%]	0.0%	0.0%	0.0%	0.0%	9.1%	37.9%	53.0%	
Cumulative [%]	0.0%	0.0%	0.0%	0.0%	9.1%	47.0%	100.0%	100.0%

Table 11: Adjusted investment costs (current prices) for the Ghutail Pilot Structure

Economic Costs: Ghutail Revetments (constant prices, adjusted costs for pilot structure)								
Investment costs	1994	1995	1996	1997	1998	1999	2000	Total
Land								
Land acquisition						8,029,437		**8,029,437**
Land lease (temp.)							595,430	**595,430**
Construction								
Constr. Contract						41,390,166	66,374,814	**107,764,980**
Material						5,315,233	1,964,389	**7,279,622**
Equipment								
Adaptation/Extension								
Adapt./ext. contract								
Material								
Equipment								
Monitor./Mainten.								
Monitoring							990,225	**990,225**
Maintenance								
Other								
Consultant					16,352,120			**16,352,120**
Other								
Total	**0**	**0**	**0**	**0**	**16,352,120**	**54,734,837**	**69,924,858**	**141,011,815**
Realization [%]	0.0%	0.0%	0.0%	0.0%	11.6%	38.8%	49.6%	
Cumulative [%]	0.0%	0.0%	0.0%	0.0%	11.6%	50.4%	100.0%	100.0%

Table 12: Adjusted economic costs (at constant 2000 prices) for the Ghutail Pilot Structure

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Attachment 2

Net Present Value and
Economic Internal Rate of Return for
Standard Protection Structures

ATTACHMENT-2

LIST OF TABLES

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Economic Analysis of Cash Flows: Standard Groyne Field (Loading Class 3)

(constant year 2000 prices)

Assumptions structure length:	1,000 m	land price:	200,000.00 Taka / ha	200,000.00		
annual erosion rate:	93 m	production:	50,000.00 Taka / ha/ yea	50,000.00		
annual increase of benefits:	4%		4%	**EIRR :**	12.11%	
				NPV (12%):	2,500,193 Tk	

Period	Project costs		Prevented Damage				Current Prices	Constant Prices
Year	Investment	M&M [1]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [2]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
0	88,571,175	0.00					-88,571,175	-79,081,406
1	80,657,500	0.00	10.16	2,114,299	528,575		-78,014,626	-62,192,782
2		4,225,000	11.89	2,573,062	1,192,983		-458,955	-326,675
3		4,225,000	13.62	3,065,142	2,006,988		847,130	538,366
4		4,225,000	15.35	3,592,472	2,985,385		2,352,857	1,335,074
5		4,225,000	17.08	4,157,084	4,144,072		4,076,156	2,065,107
6		4,225,000	18.81	4,761,117	5,500,114		6,036,231	2,730,484
7		4,225,000	20.54	5,406,822	7,071,824		8,253,645	3,333,509
8		4,225,000	22.27	6,096,565	8,878,838		10,750,403	3,876,703
9		4,225,000	24.00	6,832,836	10,942,201		13,550,037	4,362,749
10		4,225,000	25.73	7,618,255	13,284,452		16,677,707	4,794,442
11		4,225,000	27.46	8,455,575	15,929,724		20,160,299	5,174,647
12		4,225,000	29.19	9,347,691	18,903,836		24,026,526	5,506,260
13		4,225,000	30.92	10,297,647	22,234,401		28,307,048	5,792,183
14		4,225,000	32.65	11,308,644	25,950,938		33,034,582	6,035,295
15		4,225,000	34.38	12,384,044	30,084,986		38,244,030	6,238,430
16		4,225,000	36.11	13,527,382	34,670,231		43,972,614	6,404,362
17		4,225,000	37.84	14,742,373	39,742,634		50,260,007	6,535,791
18		4,225,000	39.57	16,032,920	45,340,569		57,148,489	6,635,327
19		4,225,000	41.30	17,403,122	51,504,973		64,683,094	6,705,487
20		4,225,000	43.03	18,857,288	58,279,493		72,911,781	6,748,686
21		4,225,000	44.76	20,399,942	65,710,659		81,885,601	6,767,232
22		4,225,000	46.49	22,035,837	73,848,044		91,658,881	6,763,321
23		4,225,000	48.22	23,769,963	82,744,457		102,289,420	6,739,042
24		4,225,000	49.95	25,607,562	92,456,126		113,838,688	6,696,368
25		4,225,000	51.68	27,554,138	103,042,905		126,372,043	6,637,162
26		4,225,000	53.41	29,615,467	114,568,488		139,958,955	6,563,176
27		4,225,000	55.14	31,797,616	127,100,631		154,673,247	6,476,056
28		4,225,000	56.87	34,106,952	140,711,394		170,593,346	6,377,338
29		4,225,000	58.60	36,550,158	155,477,390		187,802,548	6,268,459
Total	**169,228,675**		**997**	**430,011,972**	**1,354,837,311**	**0**	**1,497,320,608**	**2,500,193**

[1] Monitoring & Maintenance during implementation phase included in investment costs
[2] not considered in this assessment

Table 1: Economic cash flow for a standard groyne field (NPV and EIRR after 30 year period)

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Economic Analysis of Cash Flows: Standard Revetment Structure (Loading Class 3)

(constant year 2000 prices)

Assumptions structure length: 1,000 m; annual erosion rate: 111 m; annual increase of benefits: 4%; land price: 200,000.00 Taka / ha; production: 50,000.00 Taka / ha/ yea; 4%

EIRR : 12.06%

NPV (12%): 1,903,093 Tk

Period	Project costs		Prevented Damage				Current Prices	Constant Prices
Year	Investment	M&M [1]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [2]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
0	115,112,550	0.00					-115,112,550	-102,779,063
1	103,711,000	0.00	12.33	2,565,077	641,269		-100,504,654	-80,121,695
2		6,780,000	14.80	3,200,736	1,467,104		-2,112,161	-1,503,394
3		6,780,000	17.26	3,883,143	2,496,574		-400,283	-254,387
4		6,780,000	19.72	4,615,022	3,750,192		1,585,214	899,493
5		6,780,000	22.19	5,399,238	5,250,009		3,869,247	1,960,281
6		6,780,000	24.65	6,238,807	7,019,712		6,478,519	2,930,553
7		6,780,000	27.12	7,136,903	9,084,726		9,441,629	3,813,316
8		6,780,000	29.58	8,096,865	11,472,331		12,789,196	4,611,912
9		6,780,000	32.05	9,122,205	14,211,776		16,553,980	5,329,939
10		6,780,000	34.51	10,216,616	17,334,401		20,771,017	5,971,171
11		6,780,000	36.97	11,383,986	20,873,773		25,477,759	6,539,506
12		6,780,000	39.44	12,628,398	24,865,824		30,714,221	7,038,907
13		6,780,000	41.90	13,954,149	29,348,994		36,523,142	7,473,358
14		6,780,000	44.37	15,365,754	34,364,392		42,950,146	7,846,831
15		6,780,000	46.83	16,867,961	39,955,958		50,043,919	8,163,247
16		6,780,000	49.30	18,465,760	46,170,636		57,856,395	8,426,457
17		6,780,000	51.76	20,164,393	53,058,560		66,442,953	8,640,214
18		6,780,000	54.22	21,969,372	60,673,245		75,862,618	8,808,164
19		6,780,000	56.69	23,886,487	69,071,797		86,178,283	8,933,824
20		6,780,000	59.15	25,921,819	78,315,123		97,456,943	9,020,577
21		6,780,000	61.62	28,081,760	88,468,168		109,769,929	9,071,662
22		6,780,000	64.08	30,373,021	99,600,150		123,193,172	9,090,172
23		6,780,000	66.54	32,802,653	111,784,820		137,807,472	9,079,046
24		6,780,000	69.01	35,378,058	125,100,727		153,698,784	9,041,071
25		6,780,000	71.47	38,107,011	139,631,509		170,958,519	8,978,880
26		6,780,000	73.94	40,997,675	155,466,188		189,683,863	8,894,955
27		6,780,000	76.40	44,058,622	172,699,491		209,978,112	8,791,630
28		6,780,000	78.87	47,298,847	191,432,182		231,951,029	8,671,089
29		6,780,000	81.33	50,727,797	211,771,419		255,719,216	8,535,377
Total	**218,823,550**		**1,358**	**588,908,134**	**1,825,381,047**	**0**	**2,005,625,631**	**1,903,093**

[1] Monitoring & Maintenance during implementation phase included in investment costs

[2] not considered in this assessment

Table 2: Economic cash flow for a standard revetment structure (NPV and EIRR after 30 year period)

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Economic Analysis of Cash Flows: Standard Groyne Field (Loading Class 3) (constant year 2000 prices)									
Assumptions	structure length: 1,000 m; annual erosion rate: 137 m; annual increase of benefits: 4%			land price: 500,000.00 Taka / ha; production: 125,000.00 Taka / ha/ yea; 4%				EIRR : / NPV (12%):	12.05% / 332,161 Tk
Period	Project costs		Prevented Damage					Current Prices	Constant Prices
Year	Investment	M&M [1]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [2]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
0	88,571,175	0.00					-88,571,175	-79,081,406
1	80,657,500	0.00	15.58	8,099,988	2,024,997		-70,532,515	-56,228,089
2		4,225,000	19.33	10,454,043	4,719,508		10,948,550	7,792,962
3		4,225,000	23.08	12,983,462	8,154,153		16,912,615	10,748,272
4		4,225,000	26.84	15,698,507	12,404,946		23,878,454	13,549,276
5		4,225,000	30.59	18,609,984	17,553,640		31,938,624	16,181,101
6		4,225,000	34.35	21,729,260	23,688,101		41,192,361	18,633,332
7		4,225,000	38.10	25,068,303	30,902,700		51,746,003	20,899,343
8		4,225,000	41.85	28,639,702	39,298,734		63,713,436	22,975,704
9		4,225,000	45.61	32,456,704	48,984,859		77,216,564	24,861,667
Total	**169,228,675**		**275**	**173,739,953**	**187,731,638**	**0**	**158,442,917**	**332,161**

[1] Monitoring & Maintenance during implementation phase included in investment costs
[2] not considered in this assessment

Table 3: Economic cash flow for a standard groyne field in a rather prosperous area (NPV and EIRR after 10 year period)

Economic Analysis of Cash Flows: Standard Revetment Structure (Loading Class 3) (constant year 2000 prices)									
Assumptions	structure length: 1,000 m; annual erosion rate: 165 m; annual increase of benefits: 4%			land price: 500,000.00 Taka / ha; production: 125,000.00 Taka / ha/ yea; 4%				EIRR : / NPV (12%):	12.02% / 157,665 Tk
Period	Project costs		Prevented Damage					Current Prices	Constant Prices
Year	Investment	M&M [1]	Eroded Land	Value of Land Resources	Value of Agricultural Benefits	Others [2]	Net Cash Flow	NPV (12%) of Net Cash Flow
[-]	[Tk]	[Tk]	[ha]	[Tk]	[Tk]	[Tk]	[Tk]	[Tk]
0	115,112,550	0.00					-115,112,550	-102,779,063
1	103,711,000	0.00	19.22	9,995,700	2,498,925		-91,216,375	-72,717,136
2		6,780,000	24.67	13,340,184	5,933,928		12,494,112	8,893,062
3		6,780,000	30.11	16,936,234	10,405,344		20,561,577	13,067,254
4		6,780,000	35.56	20,798,623	16,021,213		30,039,836	17,045,410
5		6,780,000	41.00	24,942,905	22,897,788		41,060,693	20,802,625
6		6,780,000	46.45	29,385,453	31,160,062		53,765,515	24,320,789
7		6,780,000	51.89	34,143,495	40,942,339		68,305,834	27,587,581
8		6,780,000	57.34	39,235,164	52,388,823		84,843,987	30,595,592
9		6,780,000	62.78	44,679,537	65,654,260		103,553,797	33,341,551
Total	**218,823,550**		**369**	**233,457,294**	**247,902,682**	**0**	**208,296,426**	**157,665**

[1] Monitoring & Maintenance during implementation phase included in investment costs
[2] not considered in this assessment

Table 4: Economic cash flow for a standard revetment structure in a rather prosperous area (NPV and EIRR after 10 year period)

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BANK PROTECTION AND RIVER TRAINING (AFPM)
PILOT PROJECT FAP 21/22

FINAL PROJECT EVALUATION REPORT

MAIN REPORT – PART B

RIVER TRAINING (AFPM) PILOT PROJECT
FAP 22

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FAP 22 – RIVER TRAINING (AFPM) PILOT PROJECT

FINAL PROJECT EVALUATION REPORT

MAIN REPORT - PART B

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LIST OF ACRONYMS

AFD	-	Agence Française de Développement
AFPM	-	Active Flood Plain Management
BWDB	-	Bangladesh Water Development Board
CFD	-	Caisse Française de Développement
FAP	-	Flood Action Plan
FPCO	-	Flood Plan Coordination Organization
JTWC	-	Jamuna Test Works Consultants
KfW	-	Kreditanstalt für Wiederaufbau
PWD	-	Public Works Department (datum level)
WARPO	-	Water Resources Planning Organization

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GLOSSARY FAP 22

TERM	DEFINITION
anabranched	river planform with multiple channels divided by islands or bars, which are large in relation to the channel width
bandalling	river training measure using frames with bamboo mattresses to influence the flow pattern and sediment distribution for erosion protection and navigation support
bathymetry	spatial depth distribution, usually defined as the submerged bed topography with respect to a sloping idealised water level surface for a certain specified constant discharge
bedform	form of the river bed which scales with water depth (dune) or viscous sublayer (ripple)
bed topography	spatial bed level distribution with respect to a horizontal datum
bifurcation	point where one channel or river splits into multiple channels or rivers
braiding	formation of a river course with multiple channels, divided by bars that have a size in the order of the channel width
char	generally any accretion in a river, in particular islets in the rivers
confluence	point where two channels or rivers join
cut-off	formation of a new channel which shortens the channel bend
fixed surface screens	surface screens the foundation of which is in the river bed
floating surface screens	surface screens installed on floating elements
float tracking	measurement of surface flow velocities and flow lines by recording subsequent positions of floating objects
ghat	quay, ferry landing
hard point	local inerodible bankline, assumed to prevent a longer river reach from lateral migration (control point in river planform development)
high water bandals	bandal structure which operates at high water stages during the monsoon season
improved bandals	bandal structure with improved foundation and height adjustable screens

TERM	DEFINITION
intelligent dredging	dredging making use of river forces to influence morphological changes by very limited but intelligently chosen dredging alignment
low cost measure	recurrent measure the cost of which is low in comparison with the nearest alternative
low water bandals	bandal structure which operates at low water stages during the dry season
measures	river training interventions such as construction of structures like revetments, groynes, vanes etc. and dredging respectively
morphology	branch of geomorphology, studying the forms of alluvial beds of water bodies and their ongoing changes by erosion and sedimentation. The term is also used as a synonym for the bed topography or the shape of a river
nodal point	relatively stable location where two channels of an anabranched river system meet
permanent measures	measures designed/used for periods up to 50 years
planform	shape of bank lines or water lines on a map
recurrent measures	measures designed/used for one season or periods up to 5 years
river morphology	branch of geomorphology, studying the forms of the river bed and their ongoing changes by erosion and sedimentation
scour	deepening of the bed by erosive action of water
sediment	solid material eroded, transported and deposited by the river
stable	an alluvial river is "stable" when its bed remains at the same level because the sediment entering the river equals the transport capacity of the river
strategies	combination of measures designed to reach all objectives formulated as part of the strategy
surface screens	structures to influence the flow pattern and sediment distribution

CHRONOLOGY OF THE FAP 22 PROJECT COMPONENT

YEAR	EVENT
1991	
May 06	Presentation of Proposal
October 14	Signing of Consulting Agreement
December 01	Date of commencement of Consulting Services (Phase I)
1992	
January 13	Start of expatriate staff deployment in Bangladesh
March 14	Technical Report No. 1 (Phase I) on pre-selection of test areas
March 19	Coordination meeting on pre-selection of test areas with FPCO
March 21	Submission of the Inception Report
March 21	Approval of pre-selected test areas
March 28	Signing of subcontract for geotechnical investigations
April 02	Signing of subcontract for topographic surveys
April 08	Signing of subcontract for hydrographic surveys
April 04 to 19	Study Tour to Europe including attendance oh 5 th Symposium on River Sedimentation, Karlsruhe, Germany
May 22 to June 06	Braided Rivers Study Tour to the Yellow and Yangtze Kiang rivers in China and the Mississippi River in USA
July 16	Submission of the Interim Report
July 26	Signing of subcontract for physical model investigations
November 11	Signing of subcontract for additional topographic and hydrographic surveys
December 03	Signing of subcontract for additional physical model investigations
1993	
January 18	Submission of the Draft Final Planning Study Report
Jan. 26 to Feb. 07	Appraisal Mission of KfW and CFD
May 05	Letter to Proceed into Test and Implementation Phase (Phase II)
June 01	Start of Test and Implementation Phase
June 21	Meeting of FAP Review Committee on Draft Final Planning Study Report
June 30	Submission of the Final Planning Study Report
November 02 to 04	Experts Discussion
1994	
February 28	Submission of Experts recommendations
June 14 to 20	Review Mission of KfW/CFD
June 18	Consultants report on the results of the Experts Discussion
August 09	Approval of Final Planning Study Report by the FAP Technical Committee
1995	
February 12 to 17	Review Mission of KfW/CFD

YEAR	EVENT
1996	
June 26 to July 03	Review Mission of KfW/CFD
September 05	Submission of Report on Extended Studies on Recurrent Measures
October 29	Proposal for location of test site at Katlamari
November 13 to 17	Review Mission of KfW/CFD
December 24	Start of construction works at Katlamari Test Site
1997	
March 02	Approval of extension of the monitoring period up to December 31, 1999
March 20	Completion of construction works at Katlamari Test Site
July 14 to 21	Review Mission of KfW/CFD
September 14	Submission of Technical and Financial Proposal for Consultancy Services and construction of low cost and recurrent measures
1998	
January 06	Approval of modified Proposal of September 1997 for Consultancy Services and construction of low cost and recurrent measures
February 07	Start of construction works at Kundarapara Test Site
July 14 to 23	Review Mission of KfW/AFD
1999	
March 01 to 07	Review Mission of KfW/AFD
December 23	Approval of extension of the monitoring period up to December 31, 2000
2000	
Feb. 26 to March 06	Review Mission of KfW/AFD
2001	
April 13 to 20	Review Mission of KfW/AFD
December 31	Submission of Final Project Evaluation Report
December 31	End of the Project

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1 PROJECT FRAMEWORK

1.1 OBJECTIVES AND METHODOLOGY

1.1.1 Objectives

The objectives of FAP 22 were to find methods of appropriate interventions in the active flood plains of the main rivers, in particular the Brahmaputra/Jamuna river, with the aim to control bank erosion and to achieve river stabilisation. Measures had to be developed which would allow (1) to control the rivers in such a way that its outer channels would no longer threaten their outer banks by erosion and (2) to reduce the width of the rivers and thus to increase the area of much needed cultivable land. An alternative to the classical approach of applying "hard" permanent measures for river training had to be developed. "Soft" or recurrent measures shall convince the river rather than force it to reduce its often threatening and unpredictable behaviour into a more gentle and predictable one. The terminology "permanent" and "recurrent" indicate the possible lifetime of the measure. Permanent means long term up to say 50 years, recurrent means short term up to say 5 years.

1.1.2 Methodology

During the Study Phase the State-of-the-Art on river training had been evaluated, which was considered as an inventory and critical review of river training measures aiming at their possible application in the braided Brahmaputra/Jamuna river.

Based on the findings of the above study, which took into consideration morphological conditions such as the westward trend of the river, its widening process and possible morphological response as well as economical, socio-economical and environmental aspects, three different scenarios for training the Brahmaputra/Jamuna river were formulated varying from river training by only "hard" measures to river training with mainly "soft" recurrent measures.

As a result of the investigations on "soft" recurrent measures, surface screens and artificial channel cut-offs were selected as most promising measures. The attractiveness of the latter was considered less than the surface screens because of the considerable higher investment costs. Two basic types of surface screens were distinguished: fixed and floating structures. Fixed screens were developments from the traditional bandalling method. The new technology of floating surface screens was studied at a prefeasibility level only and remained unproven for the unique conditions of the Brahmaputra/Jamuna river.

On the occasion of the project appraisal mission of the donors KfW and CFD in January/February 1993, the study's overall concept was agreed to by all parties concerned. However, it was decided in view of the new concepts presented in the Study Report to hold an international experts discussion and to base the further steps in river training on the experts' opinion resulting from that event.

The experts' discussion was held in November 1993. The panel of international experts strongly advised to continue the FAP 22 approach as an alternative to other river training measures. They unanimously agreed to apply recurrent measures making best possible use of the rivers own force and an early prototype testing was recommended. However, as to the sequence of the measures it was suggested to start with the implementation and testing of fixed surface screens followed by the unknown technology of floating screens.

Mb

Different types of fixed surface screens were built at two test sites in the northern part of the Brahmaputra / Jamuna in 1997 and 1998. They proved to be a suitable low cost measure for river training. The same holds also for the floating screens, simple prototypes of which were tested at the first test site. Based on the monitoring results the floating elements were further developed for the application at a second test site.

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2 RESULTS OF THE STUDY PHASE

A possible option for improving the conditions along the Brahmaputra/Jamuna river with its considerable erosion and rapid changes in the location of channels and sand banks would be by changing the planform of the river. However, instead of the classical approach to narrow the river bed by applying "hard" river training works on a large scale, FAP 22 considered a more promising approach aiming at the modification of the river's planform by applying "softer" recurrent measures, for example by closing or deferring particularly aggressive developing outflanking channels. This would not result in changing the river planform from a braided into a meandering river, but in reducing the braiding index and finally resulting in a still braided river with a reduced width.

Three different scenarios for training the Brahmaputra/Jamuna river were formulated:

Scenario 1 is based on concepts defined by the China-Bangladesh Joint Expert Team for a study on flood control and river training on the Brahmaputra/Jamuna river. The basic idea is to create hard points (revetments, groynes etc.) in the first 15 to 20 years of a long-term project along the right and the left bank at regular intervals of around 30 km to achieve so-called "nodal points". The first hard points would be constructed in order to stop the westward drift of the river on the right bank at the priority sites of FAP 1 and at one of the test sites of FAP 21. In the following 40 years a stable course of the river will be achieved by the creation of stable islands protected by revetments and by the prevention of bank erosion between the nodal points with additional "hard" river training measures. The result would be a flood channel divided by large islands (mega chars), a so-called **anabranched channel system** (see Fig. 2-1).

Scenario 2 is also based on the development of an **anabranched flood channel** with the only difference that further bank erosion between the "hard" points would be prevented by recurrent measures. Hence, in the first 15 to 20 years of such a project nodal points will be created as in scenario 1, but during that phase recurrent measures will also be introduced. It is assumed that surface screens can be useful for preventing bank erosion. However, other recurrent measures are to be analyzed in a later stage of the project. In the following 40 years some more "hard" points along the right and the left bank should be constructed together with heavy revetments at the upstream heads of the larger islands (see Fig. 2-1).

The main idea of **Scenario 3** is the creation of a **braided river with reduced width** This scenario considers that the erosion along the right bank will be controlled by "hard" points as defined by FAP 1 and FAP 21. Further erosion in between those "hard" points will be prevented by recurrent measures. It was assumed that on the right bank one surface screen system moving on a yearly basis is required and that on the left bank two locations per year might suffer from bank erosion leading to a requirement of 3 floating screen systems for preventing erosion. For the reduction of the flood channel width by closing outflanking channels two additional floating screen systems were assumed to be necessary. The mega chars are not planned to be protected and might possibly disappear whereas the accretion of land along the left bank was assumed to increase replacing ultimately the isolated mega chars by chars attached to the main land (see Fig. 2-2).

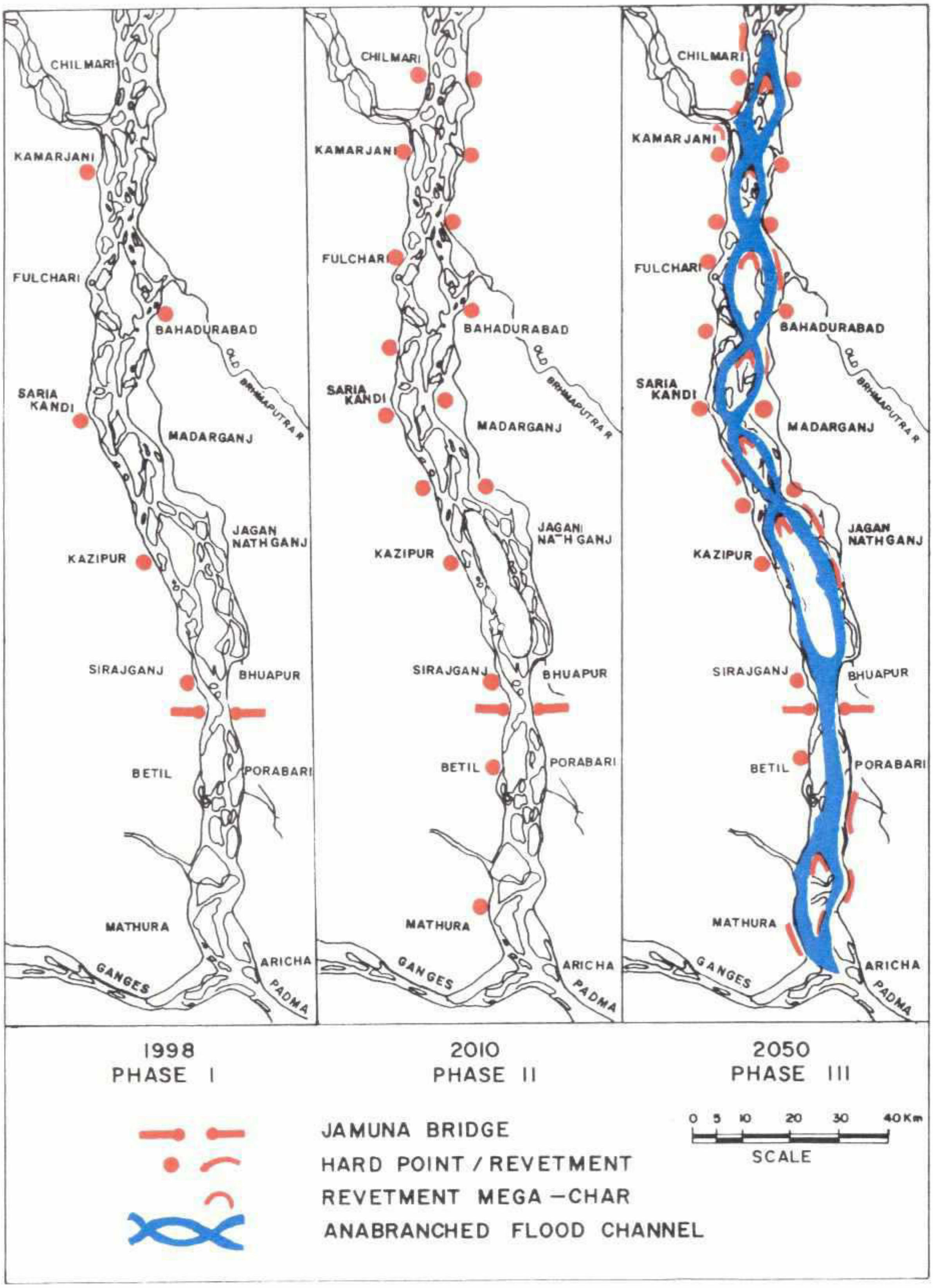

Fig. 2-1: Scenarios 1 and 2: Anabranched flood channel

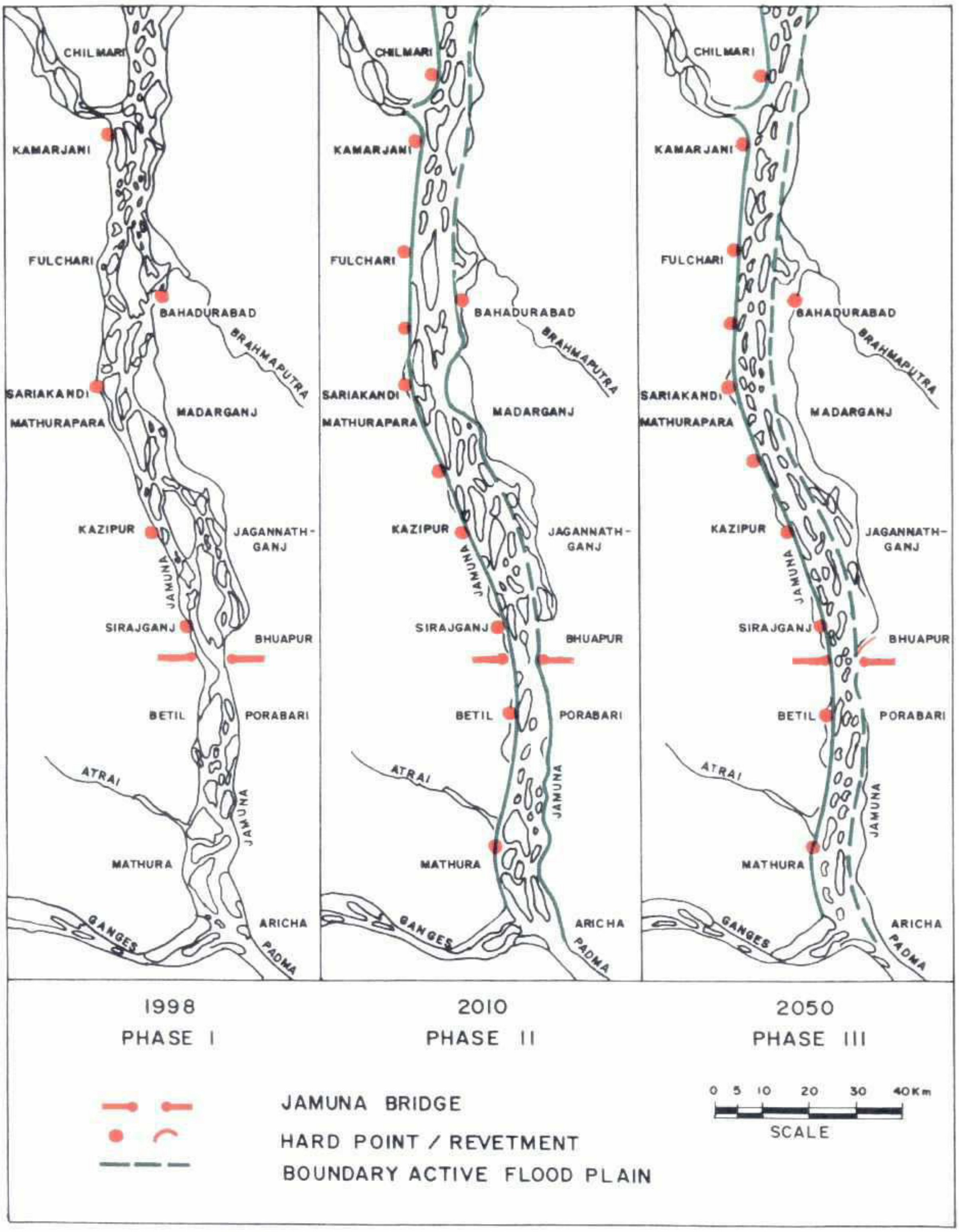

Fig.2-2: Scenario 3: Braided river with reduced width

It was stressed in the Final Planning Study Report that there is no necessity of an early selection of the scenario that would finally be implemented since the first phase of all of them is the same and consists of already planned activities or activities be decided at short notice. Only after a period of about 5 to 10 years additional measures will start to be different within the individual scenarios.

The major difference between the three scenarios is the contribution of recurrent measures. For scenario 1 no recurrent measures are planned. Although the second and the third scenario are based on the contribution of recurrent measures, the latter is drastically different, because it assumes that a major part of the river training will be done by recurrent measures and furthermore that the stable chars will gradually disappear due to a gradual reduction of the total width of the river. In fact the third scenario is the one that comes nearest to Active Flood Plain Management as far as the implementation of recurrent measures on a large scale is concerned.

The impacts of the measures of the three scenarios on the hydraulics and morphology of the river can be divided into

- changes of the river banks, and
- changes in the channel and char system.

Hence, the characteristics of the braided Brahmaputra/Jamuna river will change. However, such interference will also have an impact on the socio-economic and ecological environment. An economic analysis of the three scenarios at prefeasibility level indicated that the total cost for scenario 2 will be about 38 % lower than for scenario 1. Scenario 3, which makes the most extensive use of recurrent measures, will require a total cost about 51 % lower than for scenario 1. For details in this regard reference is made to the Final Planning Study Report.

Low cost recurrent measures are most efficient, if they are applied at specific locations of a characteristic river stretch (see Fig. 2-3):

1. along the eroding bank of an outflanking channel;
2. in an outflanking channel upstream from the eroding bank, and
3. at the bifurcation of two channels

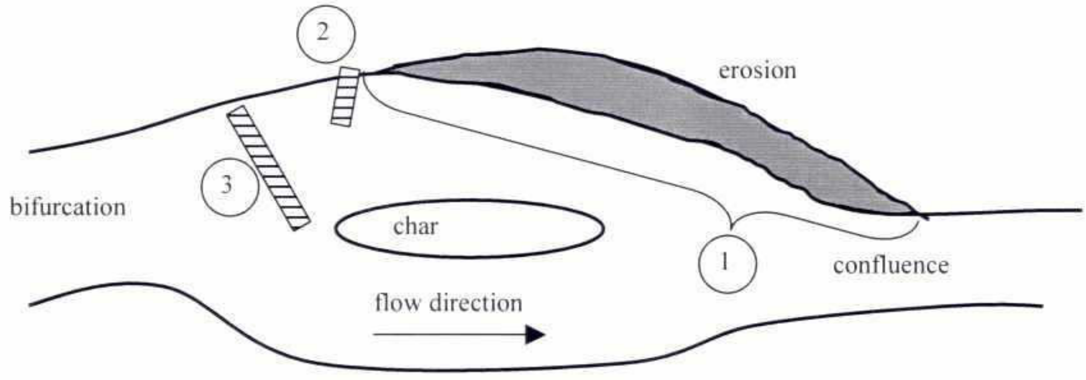

Fig.2-3: Schematic layout of bifurcation, confluence and influencing areas

As a result of investigations on "soft" recurrent measures, surface screens and artificial channel cut-offs were selected as most promising measures. Two basic types of surface screens were distinguished: fixed and floating structures. The main part of a fixed surface screen is supported by a structure of bars, beams and piles in order to maintain an opening between the screen and the river bed. In case of floating screens several screens are supported by a special barge forming a unit. Such units are connected to each other to form a row of surface screens. Each floating unit is kept at its place by an anchoring system.

Surface screens can be placed in front of an eroding bank or at an upstream bifurcation with an aggressive outflanking channel as one of the out-flowing channels of the bifurcation. The screen divides the flow in an upper flow and an underflow. The upper flow is diverted away from the outflanking channel or eroding bank and the underflow with a relatively high concentration of sediment passes the screens into the outflanking channel or towards the eroding bank. A bifurcation is selected as the most promising location. At a bifurcation a row of screens is considered while an alternative layout with a number of parallel screens is not considered as most promising for the Jamuna river conditions

The characteristic parameters of surface screens are given by the:

- orientation of the screen relative to the main flow direction;
- positioning of the screen at the bifurcation;
- position relative to the banks: connected or disconnected;
- spacing or the connections between the floating screen units;
- height and length of a screen;
- permeability of screens, and
- distance between the lower edge of the screen and the river bed.

A major advantage of the floating screens compared to fixed structures is their flexibility to be shifted after installation to the optimum location at a bifurcation and after serving their purpose at a bifurcation to be reused at other locations. Hence, floating surface screens as recurrent measures were considered technically very attractive with respect to their effectiveness and flexibility combined with relatively low costs. This is especially valid for a river as the Brahmaputra/Jamuna with its relative large and fast planform changes, which might result in expensive "hard" measures becoming useless within a short period of time.

In addition, the feasibility of artificial cut-offs was investigated. They might be considered as a specific measure along river stretches without the presence of bifurcations. Aggressive bank erosion of outflanking river bends can be prevented by either bank protection or excavation of pilot channels to achieve in due time a cut-off of the outflanking river bend. The costs of cut-off channels will be the decisive factor for such a river training measure. The investment costs are determined by the volume of material to be excavated for this single measure, which can reach considerable amounts. Therefore, the attractiveness of this measure might be considered less than the surface screen measure with its specific recurrent feature. However, in combination with surface screens it is considered as a promising method. It is to be mentioned that floating surface screens for bank erosion prevention and/or land accretion cannot always be used for scenarios 2 and 3. Sometimes other measures such as bend cut-offs are required.

3 EXPERTS DISSCUSSION

3.1 INTRODUCTION

In view of the new concept presented in the Final Planning Study Report and the significance of river training, in particular the Brahmaputra/Jamuna, it was decided in the course of the project appraisal mission in February 1993 to hold an international experts discussion and to base the further steps in river training on the experts opinion resulting from that event.

The prime objective of the experts' discussion was to elaborate fundamental and in-depth advice

- whether a river training programme of the Brahmaputra/Jamuna river should be pursued and, if affirmative,
- to recommend the further action to be taken in terms of institutional arrangements, further studies, and conditions to be met for starting the implementation.

Already in the Final Planning Study Report it was suggested that the topics for the discussion should refer to the basic technical, economic and institutional aspects allowing sound and complete recommendations on further actions to be taken.

Based on the above considerations the following topics with special regard to the unique conditions at the Brahmaputra/Jamuna river were selected for the discussion:

(1) morphological conditions and prediction methods;
(2) river training/AFPM objectives and strategies;
(3) river training/AFPM techniques and their morphological impact;
(4) economic and financial implications of river training/AFPM, and
(5) institutional requirements for river training/AFPM.

The experts' discussion took place from November 02 to 04, 1993 in Dhaka and was attended by participants from the donor agencies KfW and CFD, from FPCO and its panel of experts, from BWDB and other Bangladeshi institutions concerned.

The discussion concentrated on the technical and economical aspects of river training/AFPM techniques as well as the institutional set-up required for implementing river training works to one of the most dynamic river systems of the world. The assessment of the social and environmental impact was not discussed by the experts.

The results of the experts' discussion are summarised in "Report on the Results of Experts Discussion on River Training and Active Flood Plain Management" of February 1994.

Recommendations of the experts regarding river training/AFPM objectives, strategies and suitable techniques as well as details of a programme of action are given in the following sections of this report.

3.2 RIVER TRAINING/AFPM OBJECTIVES AND STRATEGIES FOR THE BRAHMAPUTRA/JAMUNA RIVER

3.2.1 General

A continuous study of the river behaviour is a basic requirement for identifying a possible river training scenario for the Brahmaputra/Jamuna river. Interference in such a dynamic system will include the risk that hard structures may lie idle as the river shifts away from them due to its unpredictable behaviour. Therefore, the possibility for a flexible approach for quick response was recommended for further investigation.

However, the westward drift of the river should be stopped using mainly hard structures at an early stage. As to the objectives, it was stressed that one should never call it taming or controlling the river. It is impossible to manage the river for all events. The scenarios presented in the Final Planning Study Report are only indicative and meant as a basis for sensitivity analysis. Final river training/AFPM objectives are continuously under discussion on the basis of "learning by doing".

3.2.2 River Planform

Narrowing of the active riverbed should be an objective of river training measures. The following alternative solutions for a final planform are possible:

- transition from a braided, wandering system into a more fixed but still multi-threaded anabranched system in order to keep as much as possible the natural character of the river;
- narrowing of the active bed to one single channel which is technically possible and is preferable for future requirements regarding navigation, land reclamation etc.

The decision on the envisaged final planform should be taken at a later stage and be based on gained experience in the field and increased knowledge. For the time being it was recommended that the river development strategies should concentrate on the stabilisation of the right bank using hard points together with flexible recurrent measures. The basis should be a dynamic Master Plan open for adjustments in time.

3.2.3 Bank Erosion Prevention

Complete prevention of bank erosion is technically possible but might not be desirable from an economic and ecological point of view. Moreover, bank erosion is part of the fluvial system and plays a role in accommodating the varying sediment load.

There is an optimum for the extent of preventing bank erosion. Strong erosion in between the existing and planned hard points along the right bank can be reduced. The policy might be to allow erosion at some points but to prevent overall bankline shifting. In this respect, recurrent measures were considered to be the ideal tool.

3.2.4 Applicability of Recurrent Measures

Recurrent measures can play a role in river training strategies. However, the extent to which they can be applied is unknown, because their technical feasibility had not been proven yet before the experts' discussion. Therefore, testing of recurrent measures for Brahmaputra/Jamuna conditions at an early stage was recommended.

Recurrent measures should not be considered an alternative for hard permanent structures, but should be used in combination with hard measures. They can be used

- as a tool suitable for a rapid response, which is required because of the uncertainty of attack and the possible catastrophic consequences of such an event;
- to reduce or to direct the flow attack on hard points during unfavourable conditions by guiding the flow along the structure, and

- for reducing the number of back channels, which as a result would attach mid chars to the main land. The development of attached chars is more promising from a socio-economic point of view than isolated mid chars.

3.2.5 Phases of River Training Scenarios

The following phases of river training scenarios for the Brahmaputra/Jamuna river were recommended:
- reduction of retreat of the right bank by using FAP 1 and FAP 21 priority and pilot schemes;
- development and testing of recurrent measures for closing back channels, for suppressing undesirable channels, for mitigating bank erosion and for combined application with hard points;
- updating of the FAP 1 master plan to incorporate recurrent measures.

The interaction between hard and recurrent measures were recommended to be investigated thoroughly using desk studies, physical models, numerical models and comprehensive field investigations.

A river training scenario should be flexible as even the objectives of river training are expected to change with time. The time horizon for river training scenarios will be very long. Large-scale adjustments of river training to major interventions into the river system are expected to vary from 50 to 100 years with an even longer period for a more or less complete adjustment.

3.3 RIVER TRAINING/AFPM TECHNIQUES

3.3.1 Introduction

The review of the river training measures had mainly been focused on erosion control as a tool of active flood plain management. Emphasizing the erosion control in outer channels in order to prevent outflanking or to promote closing by siltation, the following main methods of erosion control had been distinguished:
- redistribution of sediment and/or water flow at bifurcations;
- redistribution of sediment and/or water flow in the cross-sections of an outflanking channel;
- protection works on outer banks, and
- artificial cut-offs to release the flow attack in an outflanking channel.

The preliminary selection of the reviewed recurrent measures was based on a qualitative assessment of the individual effectiveness with respect to erosion control. The following techniques had been selected as most promising river training/AFPM techniques:

- dredging (low-cost and/or intelligent);
- revetments as recurrent structures along an outer bend;
- permeable groynes at a bifurcation or at an outer bend;
- a sill in an outer channel;
- surface screens or surface vanes at a bifurcation or along an outer bend;
- bottom screens at an outer channel, and
- a short-cut of an outflanking bend.

As to the mentioned locations, reference is made to Fig. 2-3 of this report.

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3.3.2 Experts Recommendations

As to the **river training scenarios** a distinction was made between the short-term phase of such a concept and the long-term planning aiming at the final planform of the river.

Short-term planning covers a period of some 20 years. The respective river training focuses on the reduction of the severe erosion of outflanking channels near threatened locations. Within this scope the FAP 1 priority works and the FAP 21 test structures serve as hard points protecting valuable places. It was not felt necessary to define the long-term planform in detail at this stage.

Long-term planning means describing the **final planform**. Alternatives were given in the Final Planning Study Report of FAP 22 and other studies. The final planform depends on the experience gained from the success and behaviour of the FAP 1 and FAP 21 structures as well as further structures built on the right bank of the river. The functioning of hard points can be improved and guaranteed with a higher level of safety if recurrent river training techniques are used to prevent unwanted channel development upstream from those hard points. Hence, the incorporation of recurrent river training techniques in the Master Plan for the Brahmaputra/Jamuna river was recommended. Another influencing factor was mentioned viz. the possibility of developments in the upstream countries and their impact on the inflowing hydrograph and the sediment load.

It was not considered necessary to define how the river should look like in about 50 years time, but to learn from the protection works built now as well as in near and distant future. The development of a selected scenario should continue while existing plans for bank protection are executed and the first field tests with recurrent river training techniques are performed. The approach in the development of a long-term strategy should be flexible and regular revisions were recommended.

Regarding the **river training techniques** the following measures were recommended by the experts:

- channel plugs (closure of river branches);
- improved bandalling techniques, and
- intelligent dredging.

The concept of surface screens suggested in the Final Planning Study Report of FAP 22 as most promising was also considered applicable and in general supported by the experts. However, a thorough understanding of the morphological processes is necessary and a more sophisticated model than developed during the Study Phase to estimate the effects of each measure and their combinations was felt to be needed.

Furthermore, it was recommended to investigate also other river training measures in detail, taking into account a combination of these measures in order to develop the optimum approach for the various training works. Finally, it was stressed that also the complexity of operation and maintenance should be taken into consideration.

3.4 PROGRAMME OF ACTION

As a result of the experts discussion including the contributions of other participants the following recurrent measures were considered to be promising and might be valuable complementary measures in an active flood plain management strategy:

- surface screens
  - fixed surface screens, e.g. bandalling, and
  - floating surface screens
- earthworks
  - channel plugs;
  - channel cut-offs, and
  - intelligent dredging.

It was recommended to investigate in detail the complex interaction of the different recurrent measures and their combination with hard points with a view to developing scenarios of intelligent combinations as well as scenarios for emergency measures.

The recommended measures, except the channel cut-offs, should be applied at appropriate bifurcations in order to influence the distribution of discharge and sediment transport.

4 IMPLEMENTATION OF TEST STRUCTURES

4.1 INTRODUCTION

Following the request of FPCO and the donors during the technical review mission of the latter in June 1994, a proposal for extended studies on recurrent measures had been submitted in July 1995. That proposal contained technical and financial details for consultancy services and construction of recurrent measures, the application of which was intended to assist FAP 21 to reach its objectives at its test sites.

At the same time a literature survey and desk study on suitable recurrent measures had been started together with the analysis of data from the first FAP 21 test site at Kamarjani and the preparation of the preliminary design of improved bandals and high water bandals. However, due to administrative problems the activities had to be suspended in November 1995 and were resumed in 1996 only on request of FPCO. A suitable test site had been selected at the Katlamari bifurcation north of Fulchari (see Fig. 4.1-1) taking into account that according to the Terms of Reference " suitable sites for trials have to be located in the northern reach of the Jamuna". The design and the structural analysis of appropriate measures/structures was done till end of 1996 followed by the construction of improved bandals and an earth dam in the Katlamari channel from January to March 1997. During the following monsoon season the behaviour of the structures was monitored and some tests with floating screens (surface screens installed on country boats) were performed.

For a second FAP 22 test site, which was identified early 1998 at the Kundarapara cut-off channel, about 2.5 km east of the FAP 21 test site at Kamarjani (see Fig. 4.1-1), another series of bandals for low water conditions, improved high water bandals as well as floating screens were designed. Out of these recurrent measures only the low water bandals could be tested during the monsoon flood 1998, whereas testing of the floating screens was postponed by one year in order to support/protect the construction activities at the third FAP 21 test site at Ghutail. This idea, however, could not be realised due to the unfavourable morphological development at Ghutail Test Site.

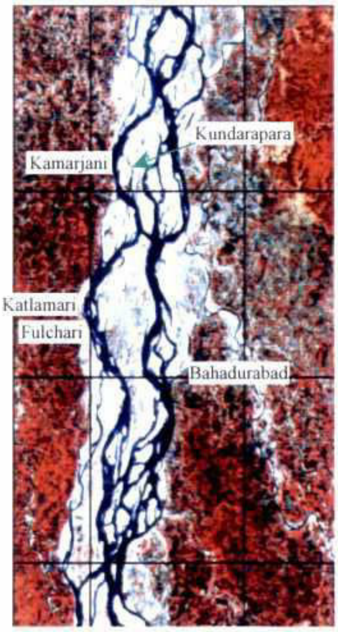

Fig. 4.1-1: Location of test sites

4.2 FIXED SCREENS AND EARTH DAM

4.2.1 Site Selection

Out of a total number of 12 alternatives identified from satellite images as possible test site in October 1996, three alternatives remained after a multi-criteria evaluation. Finally, the test site at Katlamari near Fulchari ghat was chosen (see Fig. 4.2-1). In the bifurcation area of the outflanking Katlamari channel a combination of low cost recurrent measures was planned to prevent further bank erosion.

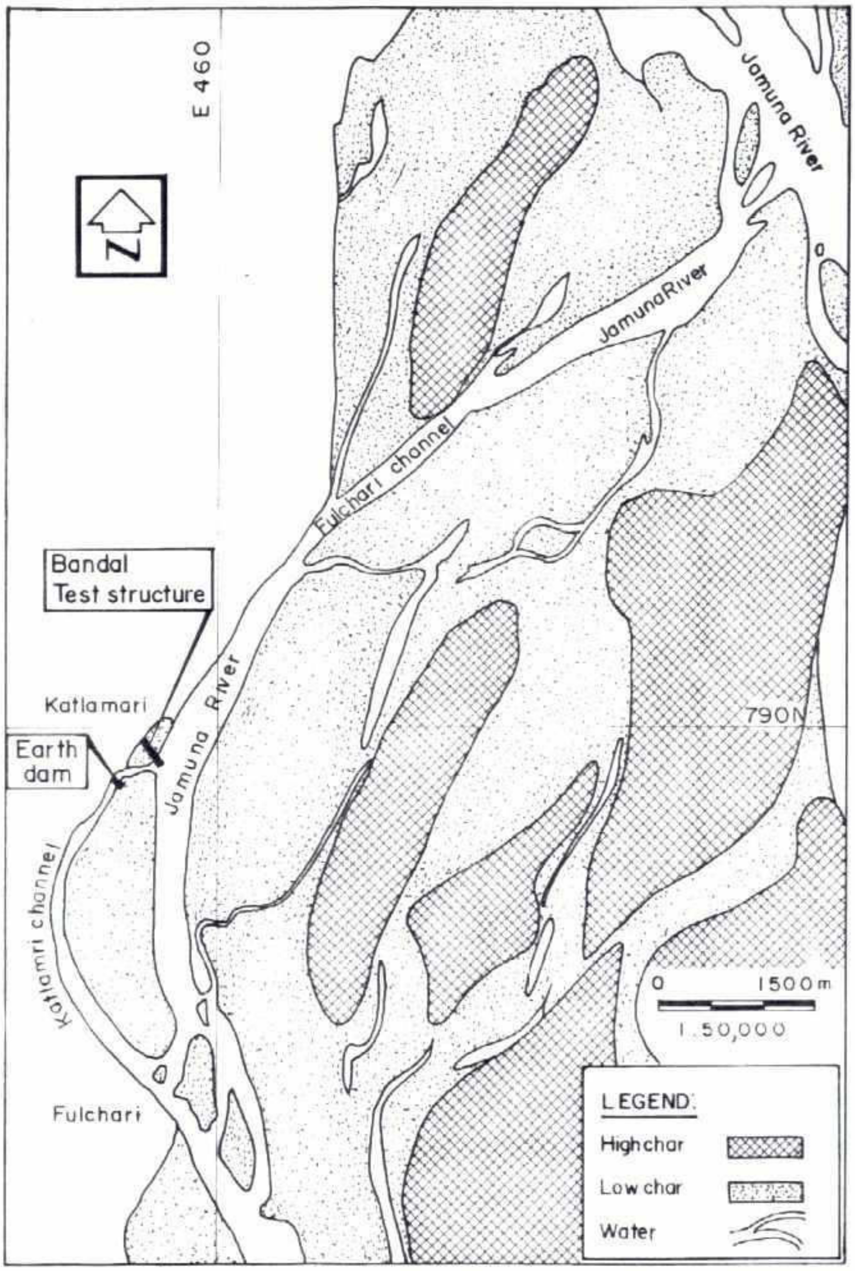

Fig. 4.2-1: Katlamari Test Site

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4.2.2 The Structures

A combination of two recurrent measures had been built (see Fig. 4.2-1):

- improved bandals at the off-take of the Katlamari channel with the aim to deflect the flow and to create sedimentation behind the structure, and
- an earth dam as a channel plug 600 m downstream from the bandal structure with the intention to close the Katlamari channel at the beginning of the following monsoon season.

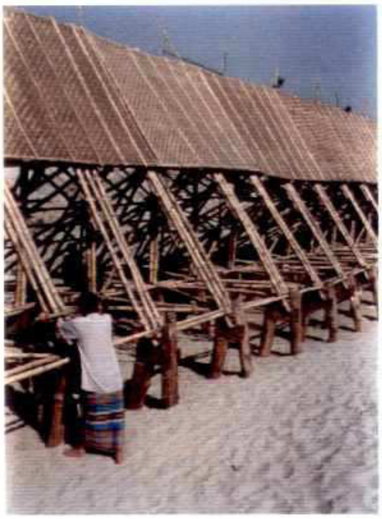
Improved bandals are structures basically of the same materials and techniques as traditional low water bandals, however, with an improved stability for higher flow velocities and installed in channels with moderate water depth up to 4 m. Their application is the same as for high water bandals, i.e. during flood flows. The latter, however, are for deep channels e.g. 10 m depth. Thus, other material is required such as steel frames and hence they are much more costly than improved bandals.

The total length of the bandal structure was 210 m with an orientation of 45° towards the expected flow of the river. The whole structure consisted of 4 main components, each of 4 m height above the char level and 45 m length (structure Type A to D) followed by two components of 15 m length each and heights of 3 m and 2 m respectively (structure Type E). The clearance between the char and the screens of the main bandals was 2 m in all cases, whereas the top of the screens was at the expected mean flood level, thus closing the upper 50 % of the cross-section. The screens were built of bamboo mats, in some sections of single wickerwork and of double wickerwork respectively. This allowed testing the influence of the permeability of the screens on hydraulic loads, wave loads and scour depth.

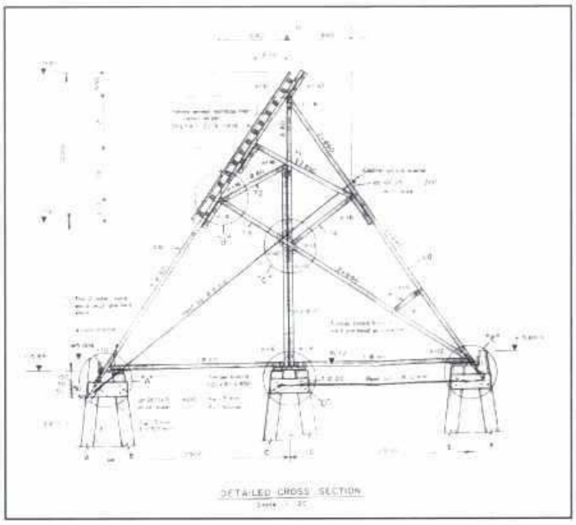
The first section of the structure close to the bank (Type A) was designed for the full hydraulic and wave loads, the next three sections (Type B, C and D) for reduced loads only to identify possible structural failures and thus the limits for the application of this recurrent measure.

The bandal structure after completion is shown on Photo 4.2-1 and some details of the different sections are given in Fig. 4.2-2.

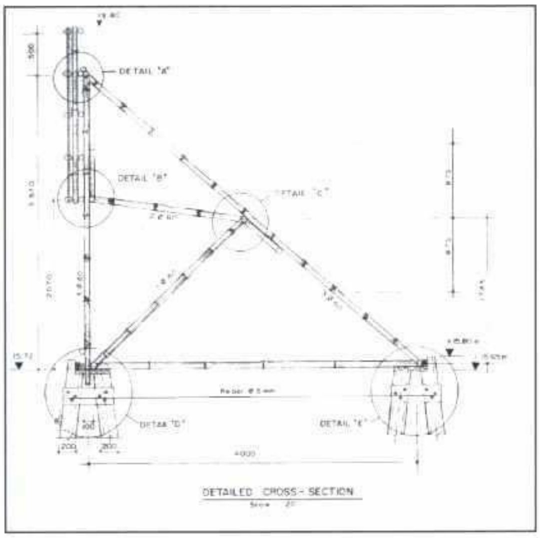
Photo 4.2-1: Improved bandal structure after construction at Katlamari in March 1997



Type A



Type B



Type C

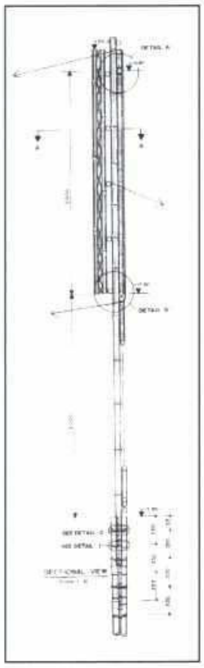


Fig.4.2-2: Details of bandal structure Type A, B and C

The foundation of the superstructure consisted of piles using bullah (tree trunks), bamboo and bamboo bundles with a spacing of the pile rows of 1.5 m in longitudinal direction. The piles were driven with an inclination of 1H:8V, which substantially improves the bearing capacity for horizontal loads.

The load assumption for the structure was based on measurements in the river branch and flow velocities computed in a mathematical river model. The scour depths below the screens was expected to be in the range of 3 to 4 m, which had to be taken into account for the design of the pile foundation.

About 600 m downstream from the bandal structure an earth dam was built of locally available char sand at the narrowest section of the Katlamari channel. This location was selected to minimise the needed quantities of sand and to maximise the effectiveness of the dam. The crest corresponded with the bottom level of the screens, so that the flow in the Katlamari channel was not totally blocked, thus diminishing the effectiveness of the screens.. The width of the crest was 2 m and the upstream and downstream slopes were 1V:3H and 1V:4H respectively. The slopes and the adjacent riverbank were protected by geo-jute and by gunny bags, whereas one layer of plastic gunny bags covered the crest. During lower water stages the dam plugged the flow in the channel. During higher stages it acted as an overtopped sill preventing high flow velocities along the bankline.

The situation near completion of the earth dam is shown on Photo 4.2-2.

Photo 4.2-2: Earth dam closing Katlamari channel during the final stage of construction in March 1997, in the background the bandal structure

The combination of these two low cost recurrent measures was such that the earth dam was closing the whole flow in the Katlamari channel until the bandals dipped into the water at rising water levels. Thereafter the bandals were activated in reducing the inflow and inducing sedimentation in the downstream area.

Photo 4.2-3 and Photo 4.2-4 demonstrate that the principle of low cost in accordance with the project philosophy and the recommendations of the experts had been observed. The first one shows the process of pile driving, the latter the construction of the earth dam by head basket method.

Photo 4.2-3: Pile driving for the foundation of the bandals in January 1997

Photo 4.2-4: Construction of earth dam in February 1997

In order to gain some experience with floating screens two country boats were used in July 1997 to study the possibility of making use of these locally available boats for influencing the flow distribution (see Photo 4.2-5). The boats were tied together and between them 5 bamboo screens of 3 m x 3 m each were installed, which could be adjusted vertically.

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Photo 4.2-5: Floating screen test in the Fulchari channel with country boats near the bandal site at Katlamari, July 1997

4.2.3 Technical and Functional Evaluation

4.2.3.1 Results of Monitoring

During the monsoon season 1997 the morphological development in the test site area as well as the impact of the structures on the river and their behaviour were monitored following a defined programme more or less daily. The following measurements and observations have been performed:

- water levels;
- waves;
- sediment distribution;
- discharge;
- erosion-sedimentation process;
- scouring process at the bandal structure, and
- structural behaviour of the bandals.

The flow measurements showed that during the monsoon 1997 the average flow velocity in the Katlamari channel was almost half of the average flow velocity in the Fulchari channel. But the sediment concentration was higher in the Katlamari channel than in the Fulchari channel. Hence, the flow entering the Katlamari channel caused deposition in the channel downstream and to the adjacent char. Moreover, the bifurcation of the Katlamari channel shifted downstream resulting in erosion of the upstream part of the Katlamari char, but the gradual migration of the Fulchari channel eroded the upstream eastern part of the char.

Hence, it can be stated that the larger scale morphological impact of the bandal structure was considerable. The scour development at the structure itself was very fast reaching maximum rates of 2 m per day and almost 3 m within four days. After that only smaller changes were observed in the range of maximum 0.5 m per day following water level changes. The maximum scour depth was about 5 m below the initial ground level. However, downstream from the bandal structure sedimentation of one to two metres was recorded in the period from June to September 1997.

During the monsoon season 1997 the Fulchari channel shifted to the west eroding parts of the Katlamari char and in September the two end sections of the structure Type E were lost due to this erosion process. The main result of the erosion-sedimentation process in the area of the bandals and downstream from them between June 14 and September 22, 1997 is shown in Fig. 4.2-3.

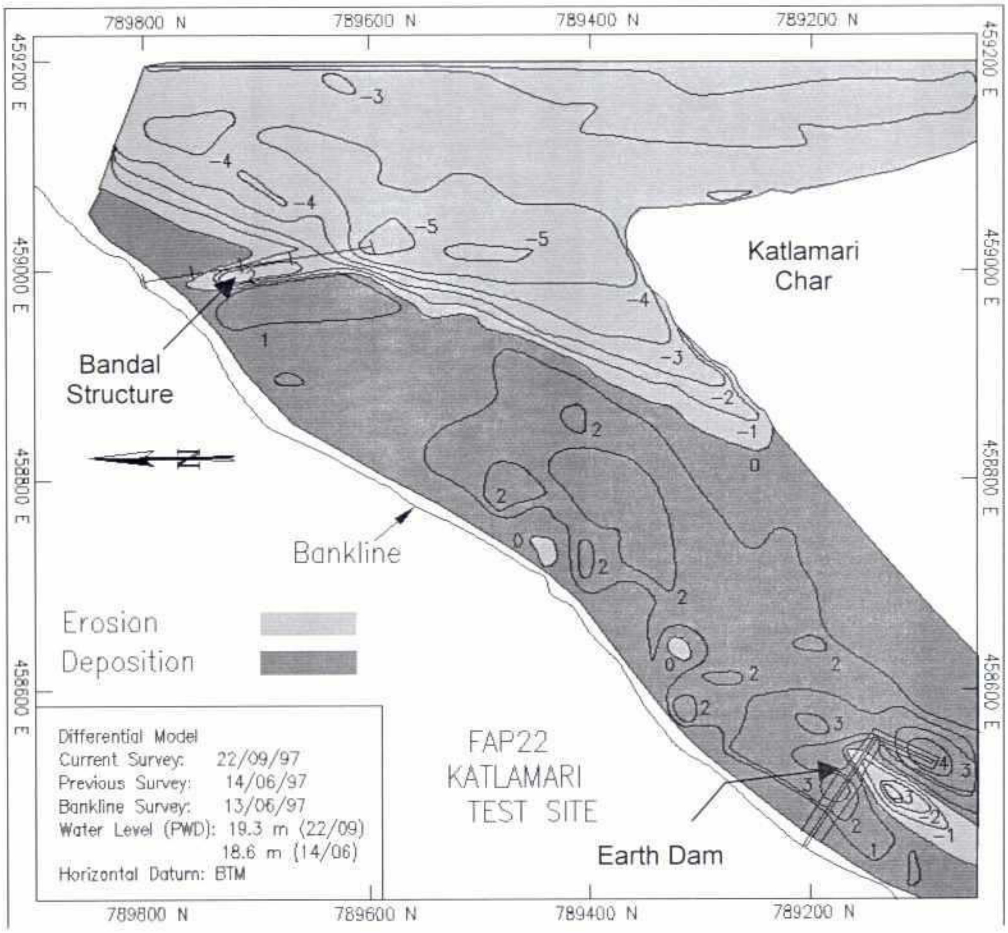

Fig. 4.2-3: Erosion and sedimentation in the area of the test structures during the monsoon 1997

The evaluation of erosion and sedimentation in a selected area downstream from the bandal structure revealed that about 85,000 m³ of sediment were deposited within one month period from June 14 to July 15, 1997. Finally, the total volume of accumulated sediments was after some fluctuations about 110,000 m³ on September 09, 1997. Details of this development are given in Fig. 4.2-4.

The earth dam was overtopped on June 10, 1997 resulting in erosion at the char side and finally creation of a channel. After about one and a half days and erosion of about 35 m of the dam an equilibrium was reached. The remaining section of about 85 m was stable during the rest of the flood season almost without any changes in length. However, since the sand had not been compacted during the construction of the dam, settlements of about 0.5 m were measured during the monsoon season. The maximum flow velocity on the crest was 1.6 m/s, but most of the time below 1.0 m/s.

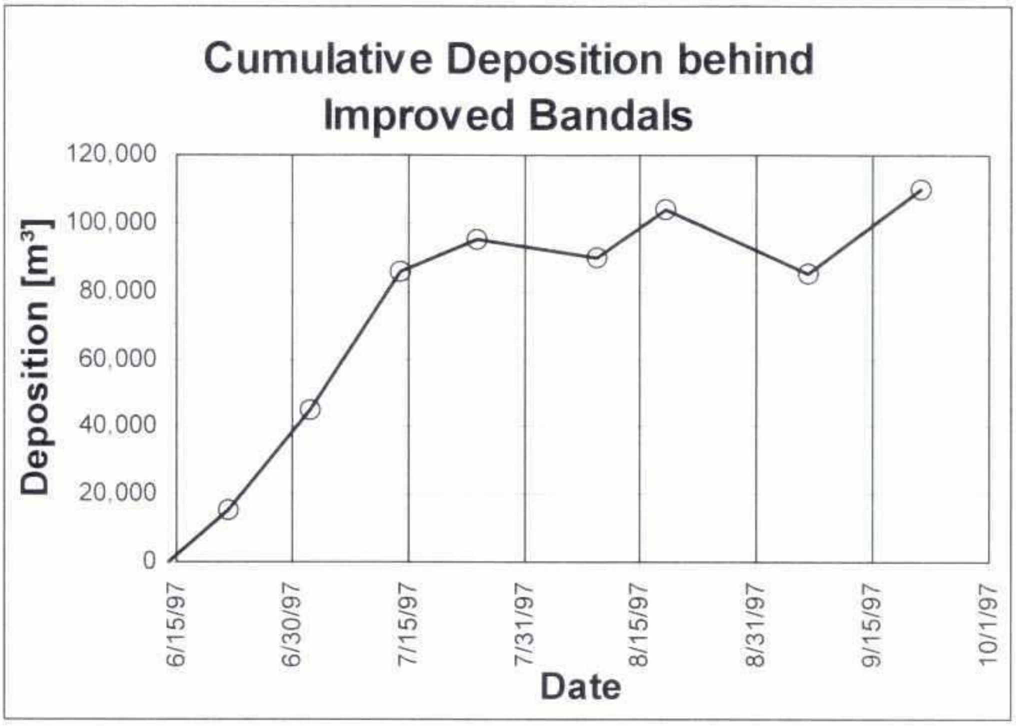

Fig. 4.2-4: Development of sedimentation downstream from the bandal structure

4.2.3.2 Performance of the Structures

The construction of the **bandal structure** did not create any difficulties. Even driving of battered piles could be done without any problems during the construction window although a high number of piles was required. Driving of bundled bamboo piles was also possible, however in an upside-down position in order to avoid breaking of the poles during the driving process.

All sections of the bandal structure survived the monsoon 1997 in good condition except Section E, which was finally washed away by the shifting towards the structure and eroding Fulchari channel. It can be concluded that the full load on the screens consisting of hydraulic and wave load, which were estimated for this application to be 3.7 kN/m and 7.3 kN/m respectively, did not occur. This can be explained by the flexibility and permeability of the structure that substantially reduced the wave loads. A reasonably safe assumption is to take into account 50% of the wave load in case of constructing fixed structures of Type A and B, or no wave load at all for structures of Type C. Nevertheless, the hydraulic load should be multiplied by a factor of 1.5 to take into account dynamic effects on the screen.

An inventory of the substructure carried out in January 1998 revealed that its condition in Section A, B, C and D was excellent. However, none of the piles in Section E was found. In some cases the superstructure i.e. the screens was slightly damaged, most probably because different types of mats had been installed, but not corresponding with the single and double pattern of the mats. In all sections the condition of the wire bindings was extremely bad. The steel was more or less totally corroded. Although the bamboo superstructure did not show any deterioration, replacement of the wire bindings was necessary. Also the metal sheets, bolts and rods for the pile head connection were heavily corroded, but replacement was not required due to adequately thick dimensions of the components.

Photo 4.2-6 shows the bandal structure during the monsoon in July 1997 and Photo 4.2-7 the structure after the flood in January 1998.The latter reveals also the scour holes below the structure. This can be seen in more detail in Photo 4.2-8.

Photo 4.2-6: Bandal structure during the flood in July 1997

Photo 4.2-7: Bandal structure after the monsoon flood in January 1998

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Photo 4.2-8: Scour below the bandal structure Type B and C in January 1998

The **earth dam** could withstand significant hydraulic loads, especially during overtopping. However, the connection between the dam and the channel bank as well as the char was damaged during the flood by erosion. A cofferdam of bamboo and gunny bags could prevent a bypass between the dam and the bank

Photo 4.2-9 shows flow measurements at the overtopped earth dam in June 1997 and Photo 4.2-10 the earth dam after the flood in January 1998, with the cofferdam at the left-hand side installed at the bank during the flood season.

Photo 4.2-9: Flow measurements at the overtopped earth dam in June 1997

Photo 4.2-10: Earth dam after the flood in January 1998

Due to the unfavourable morphological development of the Fulchari channel i.e. its shifting towards the right bank and hence endangering the structure, the test site had to be abandoned. The superstructure of the bandals was dismantled for intended use of the material at a second test site (see Section 4.3). Only Section A remained at the site and after some rehabilitation of the wire bindings the structure survived also the following flood season 1998. However, at the beginning of the monsoon 1999 the structure failed due to decay of the bamboo superstructure. The pile foundation was still in a good condition, even after the 1999 flood, but was finally taken out and reused by local people.

4.2.4 Conclusions

The scouring process below and behind the bandal structure was about 25 % more than the computed value. Smaller loads than expected and the higher structural flexibility compensated the effect on the bearing capacity of the pile foundation. A reduction of loads for the design seems appropriate resulting in less costly structures of Type B or C, which are recommended for future use. Movable, i.e. height adjustable screens, would result in a higher efficiency as the structure can start to operate already at lower water levels. However, in combination with other measures, such as a plug (earth dam), fixed screens seem to be sufficient. Also from this aspect structures Type B and C are recommendable, because Type C cannot be provided with movable screen.

The cost relationship of Type A : B : C was 1 : 0.7 : 0.6. Type C consists only of bamboo, also for the pile foundation. Due to its simplicity it is fastest to be built. Considering further cost advantages from re-using the ropes of Type C, this is the cheapest type and recommended for applications with fixed screens, provided pilferage of the nylon ropes can be prevented. Type B is less sensitive but more expensive and difficult to execute due to a foundation making use of expensive and ecologically unfavourable bullah piles.

Even though the smaller structures (Type E) designed for a smooth transition at the end were lost due to the approach of the deep channel, their future use can be recommended. The sensitivity of the

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executed measures to channel shifts is significant. In order to have sufficient safety margin also for later years of operation, it is recommended to place improved bandals further away from deeper channels that can undermine the foundation of the bandals.

The earth dam turned out to be more resistant than expected. It is of utmost importance to protect the crest (e.g. by a layer of plastic gunny bags) to increase the flow resistance for a longer period of time (as jute gunny bags decay). Nevertheless, the high cost limits the application of this type of recurrent measure. Downstream from the structure no sedimentation occurred due to an unexpected bypass channel and most probably the influence of the bandals.

Both recurrent measures alone and in combination are appropriate low cost recurrent measures suitable for the use in smaller channels, not only in the Brahmaputra/Jamuna environment when carefully designed and considering their limits.

Country boats as used for tests during the flood season 1997 are not appropriate for the application of floating screens due to their structural weakness and the high renting cost. The concept of floating screens, however, has significant advantages in terms of flexibility compared with the fixed screens. The main findings of the test with the fixed screens are summarised as follows:

- improved bandals with fixed screens proved efficient for inducing sedimentation processes;
- the pile foundation in the area of the deepest scour, however, restricts the application to more shallow river areas;
- the structural components allow only limited dimensions of the structure, though the available lengths of bamboo and wood were sufficient for the chosen application;
- structures are fixed and cannot be adapted in the horizontal plane, but adjustments in the vertical plane by movable screens seems to be possible, and
- influencing of deeper channels (or deeper parts of channels) require more flexible screens beyond the range of local materials.

4.3 LOW WATER BANDALS AND FLOATING SCREENS

4.3.1 Preliminary Remarks

After successful completion of the test works at Katlamari Test Site another proposal for Consultancy Services and the Construction of Low Cost and Recurrent Measures had been submitted in the last quarter of 1997. Additional recurrent measures were planned to be applied at Katlamari. The application of a combination of improved bandals with movable screens, another sill and a unit of floating screens with a total length of about 100 m should result in the complete closure of the Katlamari channel.

However, in January 1998 it turned out that the unfavourable morphological development in the test site area, i.e. shifting of the Fulchari channel towards the right bank and thus towards the bandal structure, continued. This trend was expected to continue also during the pre-monsoon season and the actual flood season 1998. Since already one section of the structure had been lost in September 1997 because of the limited embedded length of the pile structure, it was decided to abandon Katlamari Test Site and to identify a new one suitable for the application of the above mentioned measures.

4.3.2 Site Selection

In the course of the test site selection in 1996 twelve alternatives had been generated and investigated, among them the at that time so-called Kundarapara cut-off. This channel, which started to develop in 1996, was located about 2 km east of the Groyne Test Structure of FAP 21. The morphological development since then and the actual situation was investigated in January 1998. Finally, this location was selected as second test site of FAP 22 for the construction and testing of recurrent measures. This decision was mainly guided by the morphological development and the situation in January 1998 downstream from the groyne field at Kamarjani. In the stretch between Syedpur and Balashi Ghat severe erosion continued also after the monsoon season 1997. The intention was to promote the development of the Kundarapara cut-off channel by suitable recurrent measures and thereby to reduce the flow and thus the erosion downstream from the groyne field resulting in a benefit for the whole river stretch.

In addition, another char developed at the entrance early 1998 and separated the Kamarjani channel and the Kamarjani bypass.

The morphological situation and channel configuration of January 1998 is shown in Fig. 4.3-1.

4.3.3 The Structures

One of the main ideas in the course of selecting suitable recurrent measures, which had to be of course again low cost measures, was to extent the time of this application as much as possible, i.e. to start with the construction and their application early during the dry season and to continue as long as possible during the monsoon season.

It was decided to construct and test three types of recurrent measures taking into account the experience from the first test site and to use as far as possible material, especially bamboo, from the partly dismantled bandal structure at Katlamari.

The following measures the general layout of which is shown in Fig. 4.3-2 were designed, partly built and tested during the dry season and the monsoon season 1998:

- low water bandals;
- improved bandals with adjustable screens, and
- floating screens.

As to the location of the planned measures see Fig. 4.3-1.

The **low water bandals** were constructed from mid February to mid March 1998 at the Kamarjani bypass channel. The aim of this measure was twofold. Firstly, to reduce the flow in the bypass channel, which certainly would have an influence on the downstream flow pattern and thus on the erosion process at the riverbank downstream from the groyne field. Secondly, to increase the inflow into the Kundarapara channel and thereby promoting the further development of this channel. These bandals were like the traditional bandalling well known in Bangladesh, but the purpose of them was different. Traditional bandalling is used to maintain the navigation channel whereas the bandals at the Kamarjani bypass had to contribute to block a part of the flow and to increase the inflow into the Kundarapara channel.

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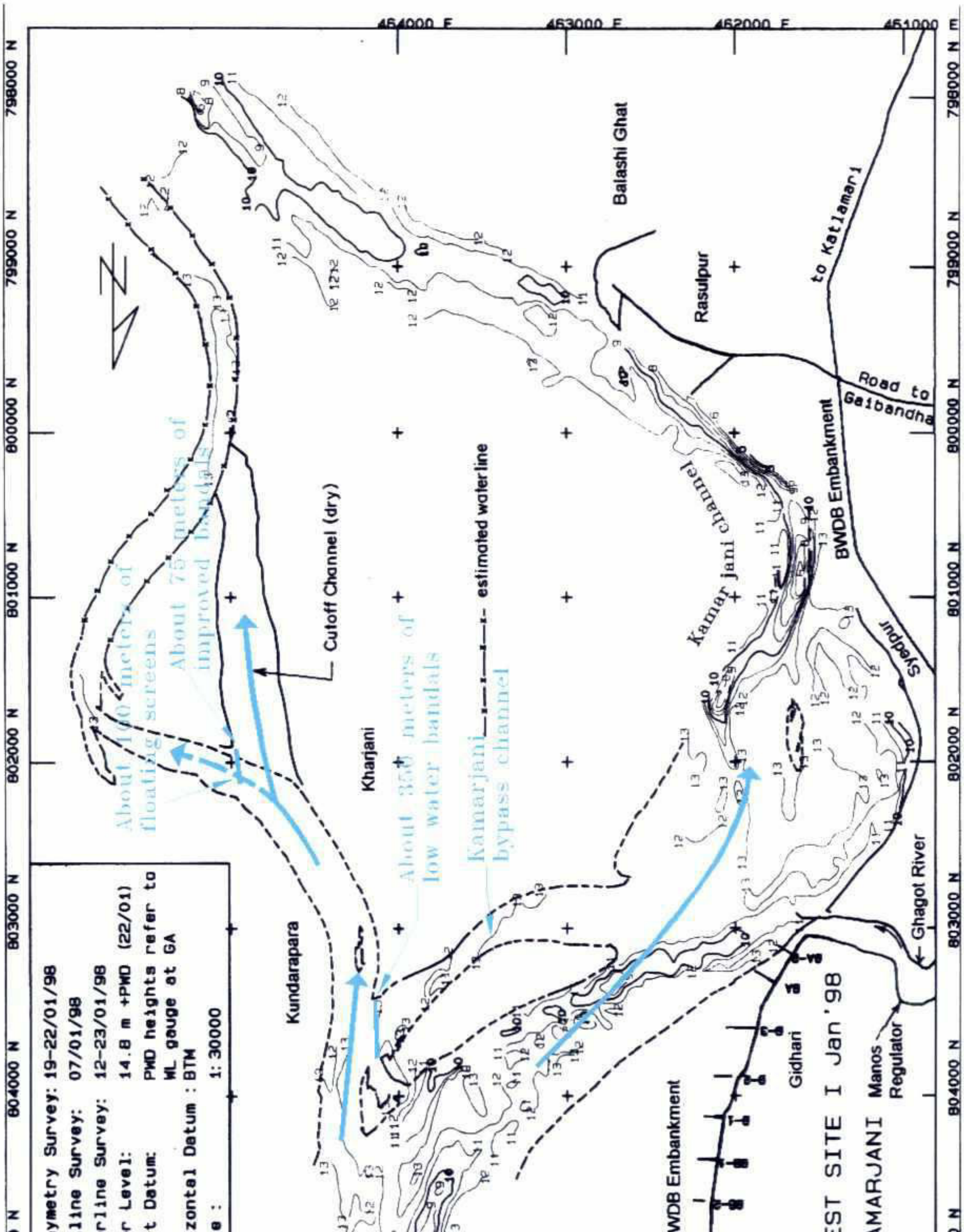

Fig. 4.3-1: Kundarapara test site and situation in January 1998

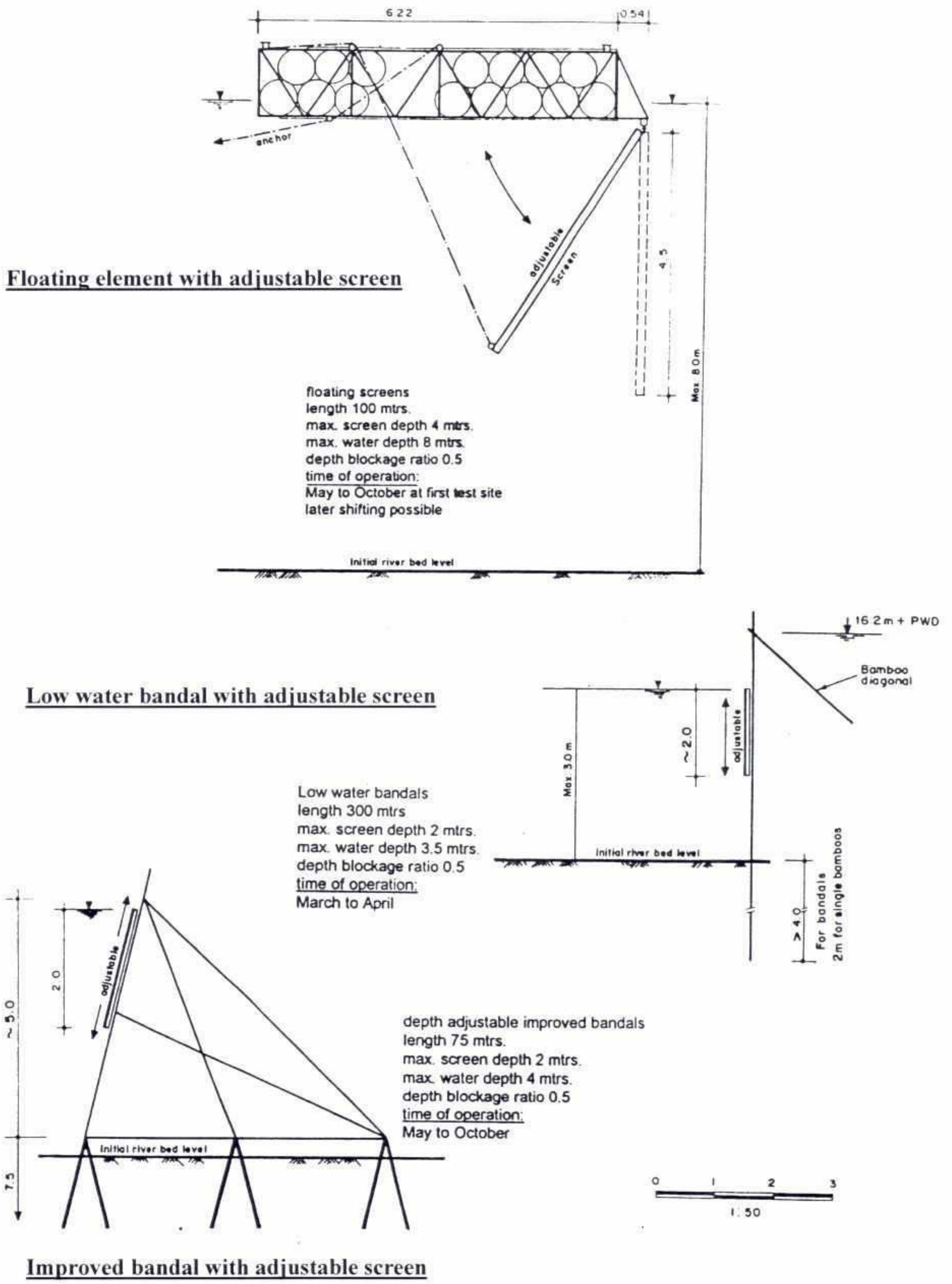

Fig. 4.3-2: General layout of recurrent measures for Kundarapara Test Site

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The low water bandals had a total length of about 350 m. The shallow areas of the cross section (see Fig. 4.3-3) were closed by single vertical piles, which were driven 2 m below the bed level. In the deeper parts with higher flow velocities stronger foundations of three bundled bamboo were driven minimum 4 m below the bed level. Driving of the piles was done from country boats (see Photo 4.3-1). The screens, which were fixed to the vertical piles, were 2 m high mattresses of different composition.

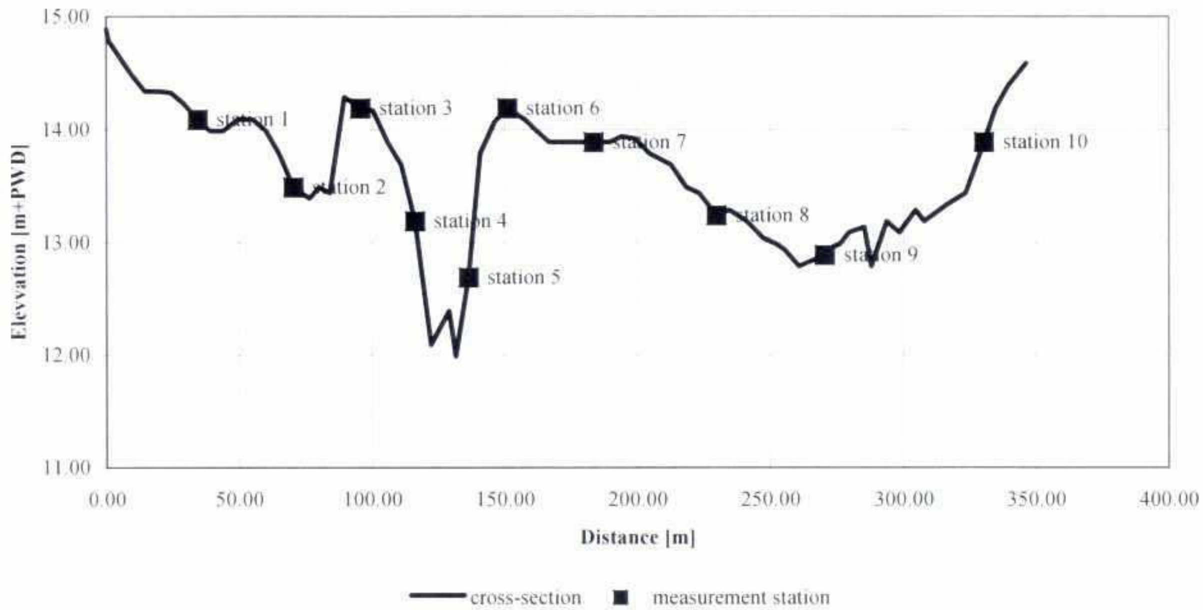

Fig. 4.3-3: Cross section at the bandal site with measurement points on March 03, 1998

Photo 4.3-1: Pile driving for installation of low water bandals

Photo 4.3-2: Low water bandals at the off-take of the Kamarjani bypass channel

The **improved bandals** had been designed with movable, height adjustable screens in order to increase the efficiency of the structure, but also to improve the control of the scour development and hence, the sedimentation process. A more detailed layout is shown in Fig. 4.3-4.

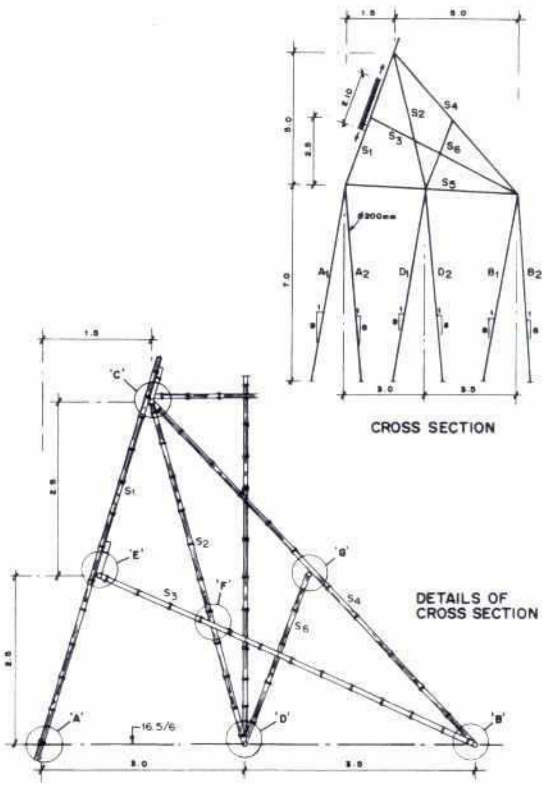

Fig. 4.3-4: Layout of improved bandals

After a late approval of the selected test site by the authorities concerned on March 03, 1998 preparation works for the foundation, i.e. the pile driving had been started, but had to be suspended since the possible effect of the structure on the morphology and channel configuration was not accepted by the local char population

The first tests with **floating screens** carried out in July 1997 in the Fulchari channel at Katlamari Test Site had shown that it is possible to fix adjustable screens to floating elements and to anchor and handle such elements. For the new test site at Kundarapara it was therefore planned to construct floating elements of about 5.7 m x 11.3 m with a maximum screen depth of 4 m. Hence, even in 8 m deep channels a blockage ratio of 0.5 could be obtained. Nine units with a total length of 100 m were planned to be installed as an extension of the improved bandals in the Kundarapara channel

Construction and assembling was started in February 1998. Since, however, the import of anchoring equipment was delayed, the elements could only be completed in June, too late to be transported to the actual test site due to the unfavourable monsoon conditions 1998, which continued over an extremely long period of time. Furthermore, the application of the floating test elements was sabotaged by local population resulting in cutting the anchor cables of the first element.

Photo 4.3-3 shows one element in the Kamarjani channel, where some anchor tests had been performed during the 1998 flood to study the loads as well as the stability of the floating elements.

Photo 4.3-3: Floating element in the Kamarjani channel in August 1998

4.3.4 Technical and Functional Evaluation

4.3.4.1 Results of Monitoring

Since the improved bandals could not be built and the construction of the floating screens was delayed, the monitoring activities in 1998 concentrated on the behaviour and impact of the **low water**

bandals on the river. Apart from the far field surveys such as bathymetric surveys, discharge measurement, float tracking etc. the following measurements had been carried out immediately at the bandal structure:

- water levels
- flow velocities;
- discharge;
- scour depth, and
- sedimentation.

As to the measurement stations see Fig. 4.3-3.

The influence of the bandals on the flow distribution had been reflected by the measurement of the water levels at a number of gauge locations along the structure. The difference of the water level upstream and downstream from the bandals was in the order of 2 to 4 cm. This difference indicates the energy loss due to the bandals.

Maximum scour rates up to 2.5 m/day had been measured, but also sedimentation rates of 1.6 m/day about 2 m downstream from the bandals. The high scour rates were recorded during water level rise. This experience has to be taken into account when low water bandals are planned to be constructed for longer use.

The structure could resist the hydraulic load and was effective till beginning of May 1998, when it was completely overtopped and bypassed just as the char separating the bypass channel from the Kamarjani channel. This holds also for the sedimentation, which was positive as long as the separating char was not significantly submerged.

An analysis of the measured data is shown in Fig. 4.3-5, which shows also the total blockage ratio that is defined as the percentage of the initial cross section closed.

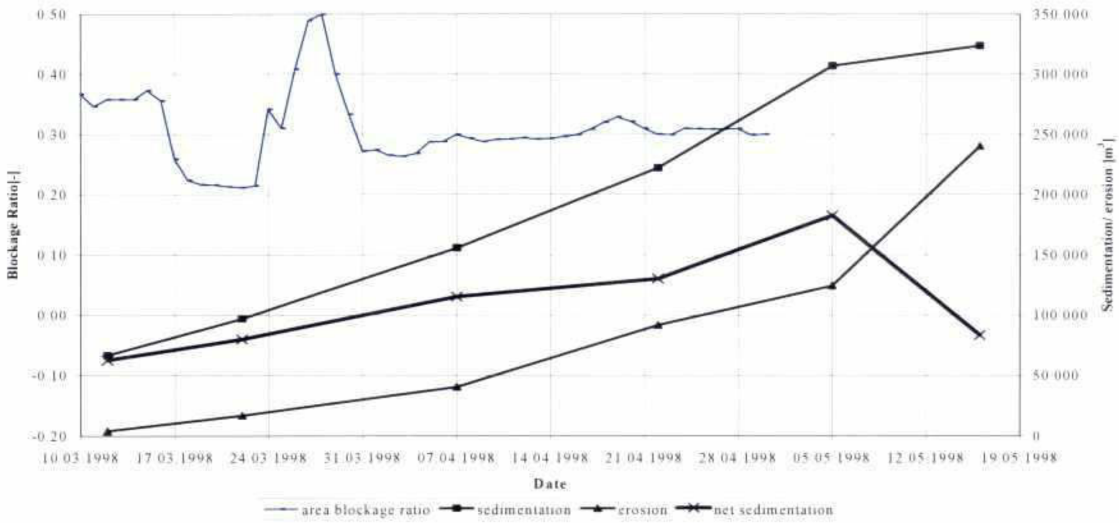

Fig. 4.3-5: Sedimentation behind the low water bandals

Photo 4.3-4: Sedimentation behind the low water bandals

4.3.4.2 Performance of the Structure

The first water level rise after completion of the **low water bandals** resulted immediately in a fast erosion, which could not be avoided quickly enough by removing the screens. This process led to erosion in the centre part of the structure and loss of piles. After repair using longer piles, the structure was stable.

The activities with the **floating screens** during the monsoon 1998 concentrated on tests with different anchors and load measurements at one test element placed in the main channel. Some tests with anchors of concrete blocks, which were also applied at Katlamari Test Site, showed that the holding power of this type of anchor results more or less only from the weight of the blocks, but not from the shape or a penetration into the riverbed. The maximum holding force was found to be about two times the weight of the anchor. Hence, a great number of concrete blocks are required, which makes the handling and anchoring process almost impossible. One test with one floating element was performed from July till mid of September 1998 in the Kamarjani channel using 2 three-ton-anchors, which were procured from ship wreckers in Chittagong. These anchors kept the floating element in position, even when the current was almost 2 m/s. The application of these anchors, however, requires heavy equipment to lift the anchors.

4.3.5 Conclusions

The application of **low water bandals** developed from the traditional bandalling technique proved to be a suitable both recurrent and low cost measure to close smaller channels by sedimentation behind the structure. However, the embedded length of the vertical piles should be more then 2 m because small water level rises induce already considerable scouring below the screens. This scouring process can even undermine the vertical piles resulting in a complete failure of the structure in the relevant area. Deeper foundation, which is increasing the construction cost only slightly, is much less sensitive to scouring. Pile driving can be done by common standard methods in Bangladesh.

It is recommended to use vertical piles of 3 bamboos, which are to be driven more than 4 m below the riverbed. The initial maximum water depth should be restricted to 3 m and the maximum flow velocity to about 1 m/s. The installation of diagonals can improve the stability of the structure considerably. Hence, the bandal structure is suitable to withstand higher hydraulic loads and thus more efficient. Moreover, structures of improved stability allow for an earlier installation during the falling limb of the hydrograph that can continue even after the dry season when the water level starts to rise. In smaller rivers with limited water depth it is even possible to maintain the structure also during the flood season taking into account partial overtopping and reduction of the efficiency.

The concept of **floating screens** is in general a technically feasible river training measure. However, whether it is in line with the idea of low cost seems to be doubtful. A suitable anchoring system must still be developed and shifting of the floating elements seems not to be possible without the help of appropriate tugboats. In particular in major channels with high flow velocities heavy anchors and other necessary equipment are required. This necessity, however, will increase the cost considerably.

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5 CONCLUDING REMARKS

In the Final Planning Study Report (1993) FAP 22 had presented possible strategies for medium and long-term training of the Brahmaputra/Jamuna river. Three scenarios for the construction of hard points along both riverbanks had been developed (see Chapter 2 of this report), which differ in the contribution of soft recurrent measures. Regardless of the latter aspect it was stressed that the suggested measures during the first phase of all scenarios would be the same. Only after a period of 5 to 10 years the additional measures would be different within the individual scenarios.

Eight years after presentation of these strategies a number of hard points have been constructed on the right bank of the river and the left bank as well. They are shown in Fig. 5-1, which compare with Fig. 2-1 and 2-2 respectively. This present network of hard points includes heavy structures designed under FAP 1 for a return period of 100 years, more cost-effective bank protection structures designed and built under FAP 21 as well as structures implemented by BWDB. However, the experience with the on-going erosion along both riverbanks shows that the network of hard points has to be improved by additional structures during the coming decades. On the one hand some of the structures were damaged or failed at least partly and on the other hand the theoretical spacing between the structures is obviously too large.

In the course of the required long-term programme for the stabilisation of the river course, the application of recurrent measures on low cost basis, for supplementing and completion of the hard point network should be considered now. Although the actual field test programme on recurrent measures under FAP 22 was very limited, quite encouraging results could be achieved.

The application of low water bandals during the dry season can contribute to close smaller secondary channels. Their use can even be extended if equipped with height adjustable screens instead of fixed ones. This would allow an earlier installation and operation already during the falling limb of the hydrograph. Moreover, the screens could also be adjusted to the rising water level during the pre-monsoon season.

Further improvement of the foundation and superstructure of the bandals tested at Katlamari Site will definitely allow their use even during the monsoon season. If provided with adjustable screens, their effectiveness can be increased considerably.

Floating screens can be used at carefully selected locations e.g. in smaller channels or to support/supplement the fixed screens. In wider channels or in channels with high flow velocities their application is limited by the required anchor system and the equipment to handle the anchors. However, they are recommended to be used as a kind of moveable permeable groyne for bank protection.

Recurrent measures will definitely be suitable to contribute to the stabilisation of the Brahmaputra/Jamuna river and other major rivers in Bangladesh, but additional investigations and tests are recommended to allow for further improvement and optimisation of the structures.

Finally, it must be stressed that people's participation in the course of the application of recurrent measures is of utmost importance. The acceptance of the measures by the local affected people is indispensable. Local social conditions must be taken into account in order to prevent parts of the population to oppose the measures. Hence they have to be informed by the concerned authorities and the local politicians in time about the function of the structures and to be convinced regarding the benefits of the planned river training interventions.

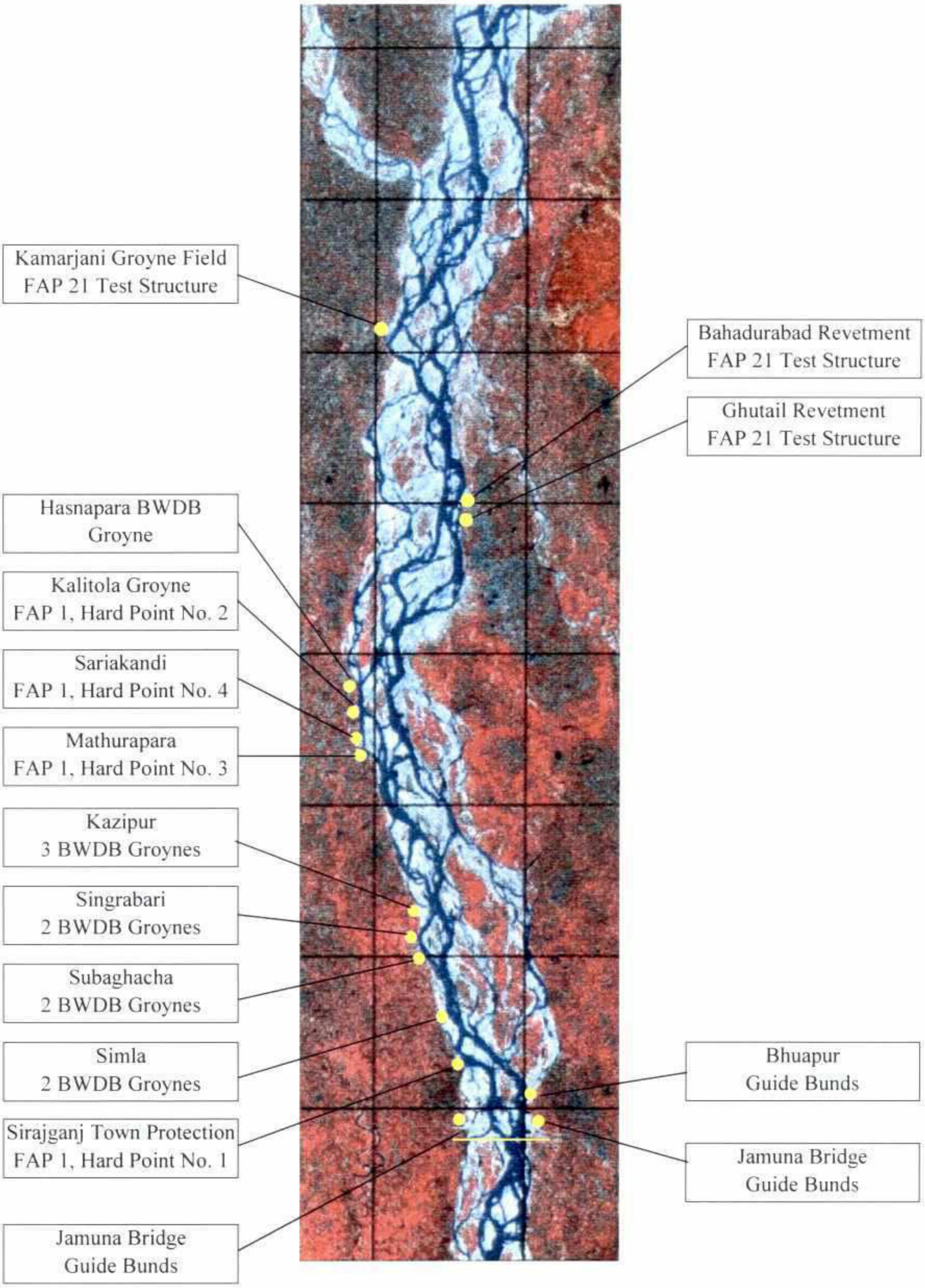

Fig. 5-1: Locations of major bank protection structures at the Brahmaputra/Jamuna river erected since 1994

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