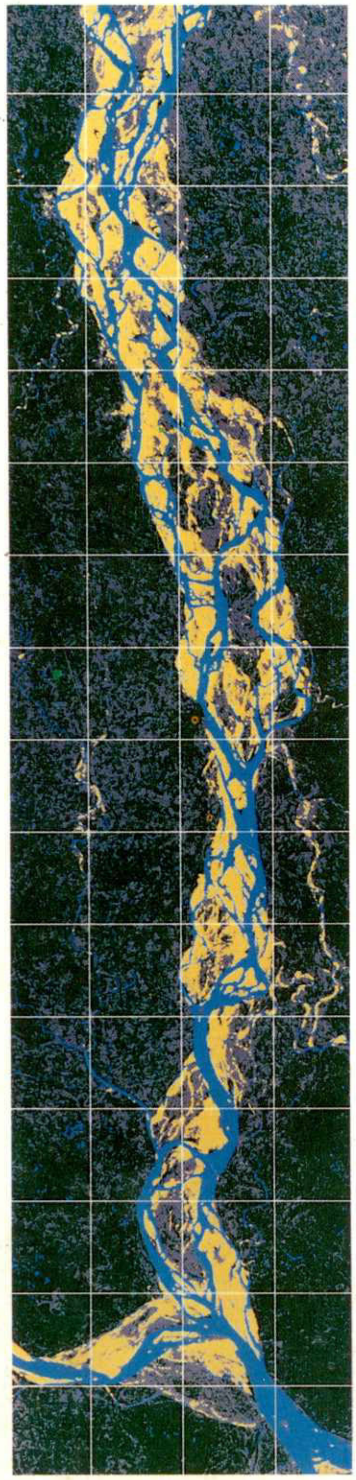
GOVERNMENT OF PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANIZATION

FEDERAL REPUBLIC OF GERMANY

KREDITANSTALT FÜR
WIEDERAUFBAU (KfW)

FRENCH REPUBLIC

CAISSE FRANCAISE DE
DEVELOPPEMENT (CFD)



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

TEST AND IMPLEMENTATION PHASE FAP 21

REPORT ON MONITORING AND ADAPTATION AT BAHADURABAD TEST SITE

MONSOON 1997

MARCH 1999

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)

2

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

TEST AND IMPLEMENTATION PHASE FAP 21

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MONSOON 1997

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

REPORT ON MONITORING AND ADAPTATION
AT BAHADURABAD TEST SITE

Table of Contents

Page

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS AND GLOSSARIES

AFPM	-	Active Flood Plain Management
BTM	-	Bangladesh Transverse Mercator
BWDB	-	Bangladesh Water Development Board
CFD	-	Caisse Francaise de Developpement
DGPS	-	Differential Global Positioning System
EDM	-	Electronic Distance Measurement
EGIS	-	Environmental Geographical Information System
FAP	-	Flood Action Plan
FPCO	-	Flood Plan Coordination Organization
G I	-	Galvanized Iron
GM	-	General Model
GoB	-	Government of Bangladesh
GPS	-	Global Positioning System
HW	-	High Water
KfW	-	Kreditanstalt für Wiederaufbau
Landsat	-	Land (Remote Sensing) Satellite
LW	-	Low Water
PWD	-	Public Works Department
RRI	-	River Research Institute
SLW	-	Standard Low Water
SPOT	-	System Probatoire d'Observation de la Terre
WARPO	-	Water Resources Planning Organization

7

1 INTRODUCTION

1.1 BACKGROUND

The Bank Protection Pilot Project is component 21 of the Flood Action Plan (FAP). It is jointly financed by Germany and France and was awarded by the Flood Plan Coordination Organization (FPCO) represented by the Kreditanstalt für Wiederaufbau (KfW) to the joint venture Rhein-Ruhr Ingenieur-Gesellschaft mbH as lead partner, Compagnie Nationale du Rhône, Prof. Dr. Lackner & Partners and Delft Hydraulics in association with Bangladesh Engineering and Technological Services Ltd. (BETS) and Desh Upodesh Ltd. (DUL).

As per Terms of Reference the Consultancy Services are to be performed in two phases, a Planning Study Phase (Phase I) followed by a Test and Implementation Phase (Phase II).

After submission of the Draft Final Planning Study Report in January 1993 a joint mission of KfW and CFD has carried out the project appraisal to proceed into Phase II of the Project. The Mission agreed to the overall concept of Phase II proposed by the Consultant the essence of which is the construction of a combination of permeable and impermeable groynes and of various types of revetments at two different test sites in two successive seasons.

The Test and Implementation Phase started on June 01, 1993 after the "Letter to Proceed" had been issued by FPCO on May 15, 1993.

The final design of the Revetment Test Structure at Bahadurabad Test Site began in September 1994 and was finalized in April 1995 based on the preliminary design and construction methods of the Planning Study, supplemented by additional studies and investigations viz. morphological studies, geotechnical investigations, physical model tests in Bangladesh and France as well as topographic and hydrographic survey. After suspension of the construction works in January 1996 and necessary modification of layout and design of the test structure, the actual execution of works started in November 1996. The structure was complete in all respects on June 12, 1997.

1.2 OBJECTIVES OF THE PROJECT

The objectives of the Project are to find improved solutions for bank protection works against erosion by designing, specifying and constructing different types of groynes and revetments using different materials and protective layers and investigating at the same time the suitability of local materials and construction methods. After construction of the test structures their behaviour is to be monitored for a period of at least three years. The final objective is to develop and optimize design criteria, cost-effective construction and maintenance methods which shall serve as future standards, most appropriate for the prevailing conditions at the Jamuna and other rivers of Bangladesh. Hence, the test structures were to be designed in such a way and with such a level of safety that certain damages of the structures are allowed, are even required, because a test work which does not suffer any damage in the course of the monitoring and adaptation period may be oversized and therefore not be suitable to identify the limits and to develop new standards.

To achieve the above objectives, regular monitoring of the test structures is a must after their completion as well as preventive maintenance and adaptation of the structures taking into account the results and observations of each monitoring period. For the development of suitable adaptation measures, however, further studies and investigations are possibly required.

2 THE STRUCTURE

2.1 INTRODUCTION

The construction of the Revetment Test Structure was originally planned about 4 km south of Bahadurabad Ghat based on the investigations during the Study Phase. Since, however, no substantial erosion occurred at the pre-selected test site, this area was abandoned and a more suitable one was selected in September/October 1995 at Kulkandi-village just downstream from Bahadurabad Ghat. The decision on the final location of the test structure was taken on October 11, 1995. However, end November it emerged that the Subcontractor could not mobilize the main construction equipment for dredging and under water works in time. Therefore, the design of the structure was modified in such a way that all components of the structure, even the falling and launching aprons could be built entirely in the dry during the lean season 1995/96. Since, however, even then the rate of progress of all works was too slow to comply with the contractual Time of Completion, it was finally decided on January 31, 1996 to defer the final completion of the test structure until next dry season.

Based on the experience in 1995 and January 1996, and after identification of the main constraints preventing the completion of Works as per original schedule, a proposal for the final implementation of the revetment test structure during the dry season 1996/97 was submitted in April 1996 taking into account the morphological analysis of the test area until then.

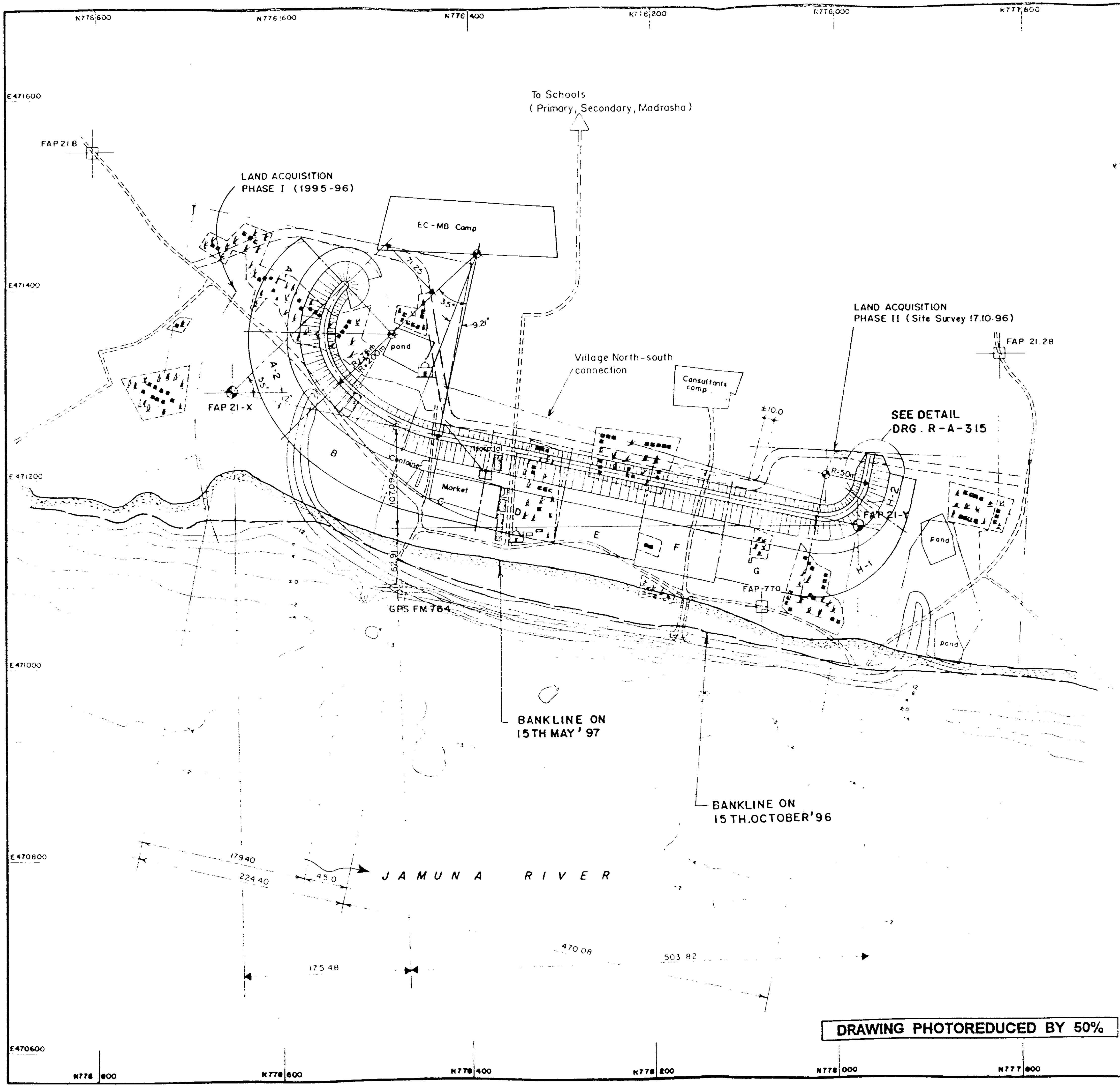
To verify that the location of the selected test site was still suitable for the revetment test structure, a further morphological analysis had been carried out in September 1996 on the basis of satellite images and survey data. This update on the morphological developments indicated that continued attack in 1997 had a high probability.

The most important prediction in March 1996 was that the eastern approach channel would excavate its bed until it would meet the western approach channel, thereby creating a confluence scour hole in front of the bank at Kulkandi around Northing 779000 and 778000. Indeed a large scour hole developed at Northing 778800 in the second half of July, while a scour hole 1200 m further downstream disappeared completely by locally more than 10 m sedimentation. The situation became more complicated in August, when a deep channel shifted towards the bank over the full length of the planned structure with severe erosion of the river bank resulting in a complete loss of the unprotected structure of 1995/96. Hence, another location of the Revetment Test Structure had to be determined and the design of the structure to be adapted accordingly. A proposal was presented in October 1996, which was discussed with the Client and the donors during a review mission of the latter in November with the final decision in favour of the location as shown in Fig. 2.1.

2.2 DESCRIPTION OF THE REVETMENT TEST STRUCTURE

The final design of the Revetment Test Structure is based on the modified design of November 1995 which allowed all construction works in sheltered conditions without any under water works. The deepest excavation level for the construction of the launching and falling aprons was fixed at 14.50 m+PWD, which is above the Standard Low Water (SLW) level of 13.30 m+PWD.

The total length of the structure is 662.5 m split up into 10 sections, each consisting of a sloped revetment, a launching apron and a falling apron. For the construction of the individual sections different material had been used for the cover layer and filter layer of the revetment as well as for the launching and falling aprons. As to the detailed layout of the structure and details of the used materials see Fig. 2.2 to 2.4. The as-built drawings of the individual sections are presented in Annex A.



CONTROL POINT SCHEDULE			
No. of Control Points	Co-ordinates BTM		Elevation (m PWD)
	Easting	Northing	
FAP21-B	471,548.482	778,800.937	+ 20.246
GPS FM764 (*)	471,086.185	778,478.880	—
FAP21.28	471,330.663	777,813.399	+ 20.139
FAP770	471,067.515	778,073.154	+ 20.248
FAP21-X	471,293.485	778,654.360	—
FAP21-Y	471,149.095	777,975.060	—

*) not existent anymore

SCALE 1 : 2000 (0, 50, 100, 200 m)

LEGEND

- Embankment
- Road Pucca, Kacha, Footpath
- High River Bank
- House Pucca, Kacha & Tinshed
- Traverse Station & Pillar
- Home stead
- Mosque

NOTES

1. The topography shown on this drawing is based on J.T.W.C's survey of Oct '96 combined with topographical maps of Survey of Bangladesh
2. Levels refer to ± 0.00 m PWD
3. Bankline and bathymetric survey as on 09/10/96
4. Reference Drawing R-A-302 Detailed Layout of Test Structure

REV	DATE	NAME	DESCRIPTION	APPROVED
1	01.6.97	Anowar	AS BUILT DRAWING	

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANISATION (WARPO)

BANK PROTECTION PILOT PROJECT FAP-21

JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE

TEST SITE II - BAHADURABAD

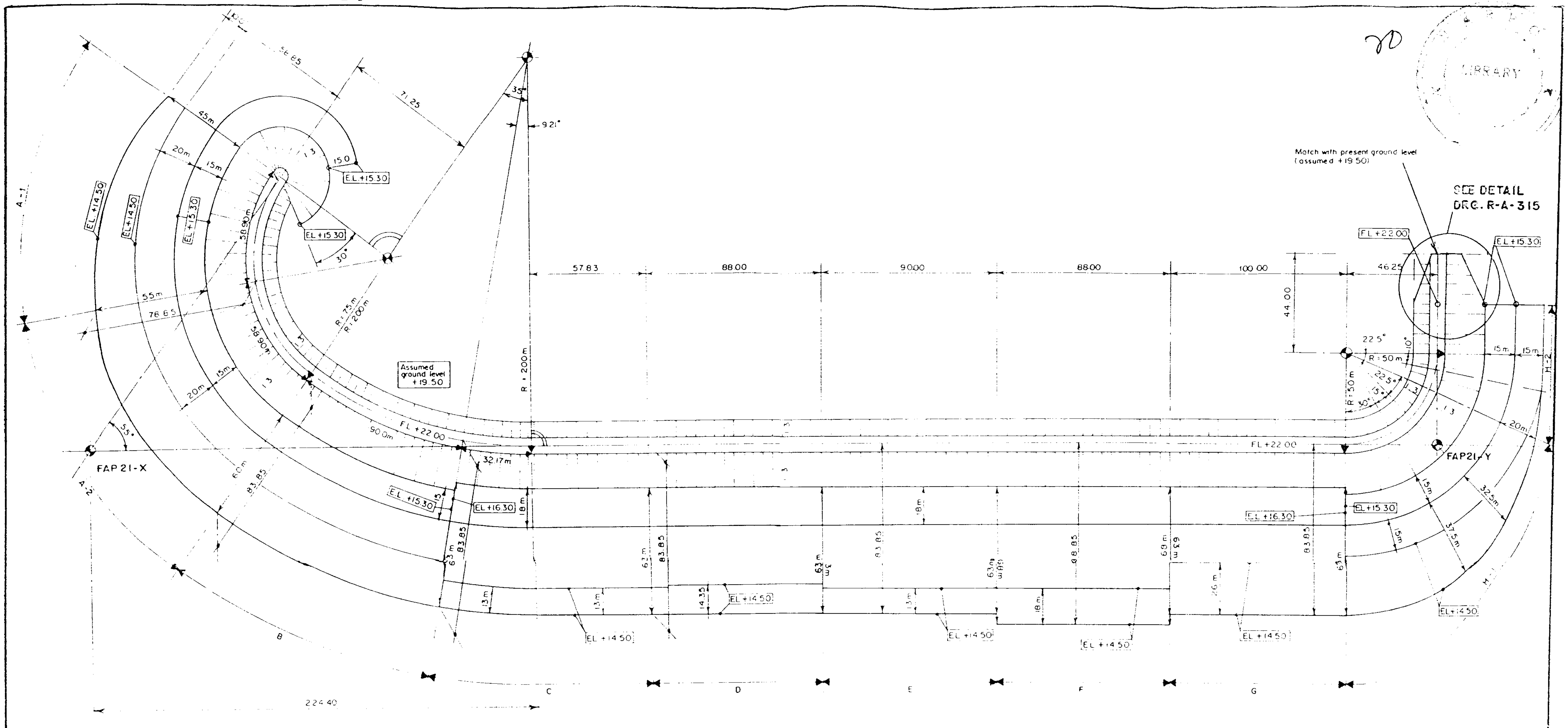
GENERAL LAYOUT OF TEST STRUCTURE
(1996/97)

	NAME	DATE
DRAWN	ANOWAR	03-11-96
CHECKED		03-11-96
APPROVED		

SCALE 1 : 2,000

FIG. 2.1

REVISION 1



CONTROL POINT SCHEDULE			
No. of Control Points	Co-ordinates BTM		Elevation (m PWD)
	Easting	Northing	
FAP 21-X	471,293.485	778,654.360	—
FAP21-Y	471,149.095	777,975.060	—

NOTES

1. Levels refer to ±0.00m PWD
2. Measurements in meter
3. F.L. : Finished level
   E.L. : Excavation level
4. Reference Drawings
   - R-A-301 General Layout of Test Structure (1996/97)
   - R-A-305 Test Structure A General Plan and Cross Sections
   - R-A-313 Test Structure H Cross-Sections H-1, H-2 Cross Sections, Details

DRAWING PHOTOREDUCED BY 50%

REV	DATE	NAME	DESCRIPTION	APPROVED
1	21.4.97	Anowar	AS BUILT DRAWING	

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANISATION (WARPO)

BANK PROTECTION PILOT PROJECT FAP-21

JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE

TEST SITE II - BAHADURABAD

DETAILED LAYOUT OF TEST STRUCTURE (1996/97)

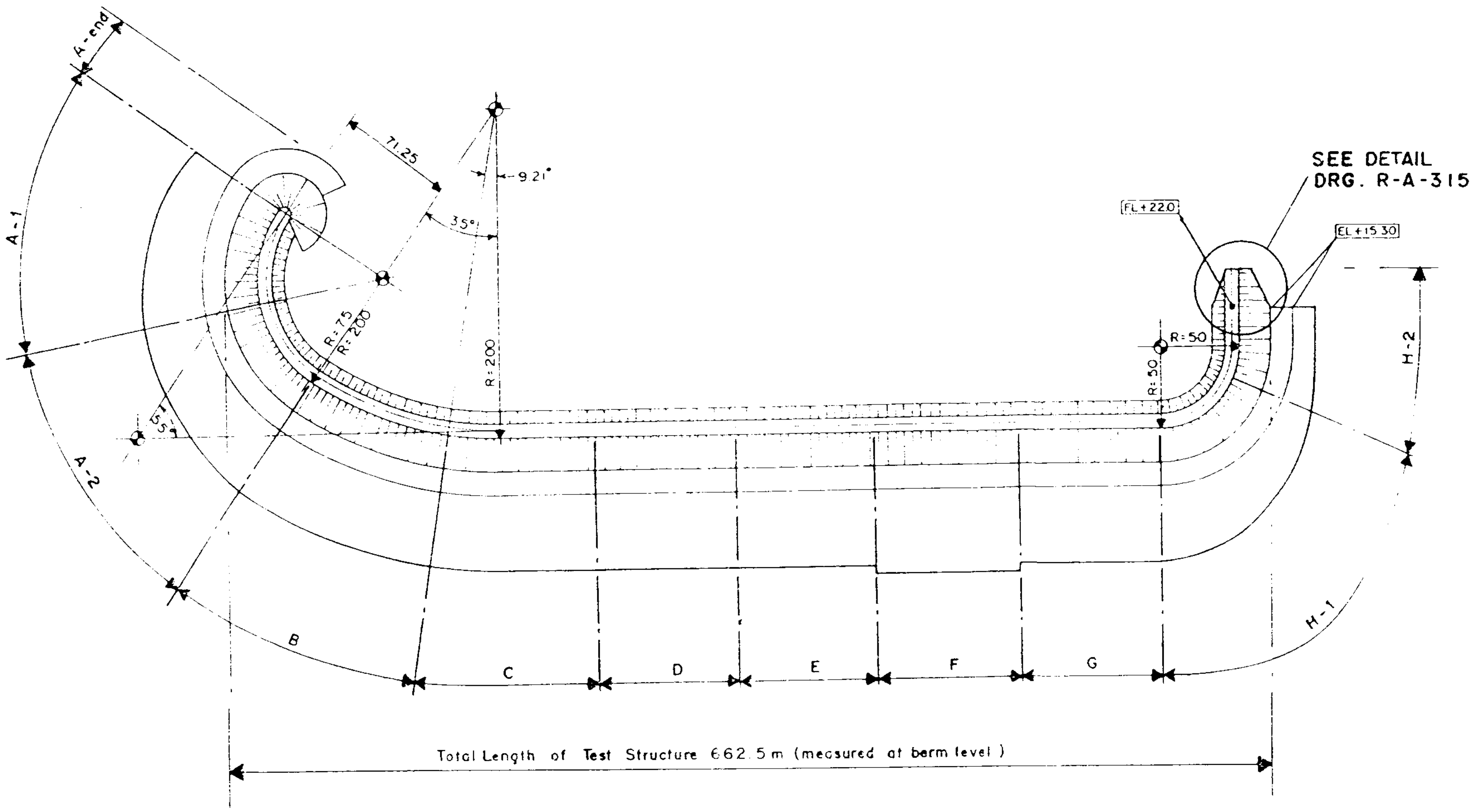
	NAME	DATE
DRAWN	ANOWAR	09-10-96
CHECKED		03-11-96
APPROVED		

SCALE: 1 : 1000

FIG. 2.2

REVISION 1

22



0 50 100 200 m

Scale 1 2000

NOTES

1 Levels refer to 0.00m PWD
2 Measurements are shown in meter
3 F.L = Finished level
E.L = Excavation level
4 Reference Drawings
R-A-302 Detailed Layout of Test Structures
R-A-304 Geotextile Filter Materials General Arrangement

	Test Structure	A - end	A - 1	A - 2	B	C	D	E	F	G	H - 1	H - 2	H-2 (end)
	Land-sided slope	Brick mattress d = 15cm	In all sections Durba grass sads laid on Geo-jute soil saver							Detail on Top of Embankment Drg.No.R-A-314			Village Road
River side	Approximate length along toe of upper slope (at berm level)	~ 87.40	~ 74.70	~ 74.70	~ 99.10	~ 93.20	88.0	90.0	88.0	100.0	~ 82.75	~ 97.60	~20.0 / ~30.0
River side	Revetment above berm level (+15.3m to +22.0m PWD)	Brick mattress d = 15cm	Brick mattress d = 15 cm	Wiremesh mattress d = 23/36 cm with stone fill Grade B (D_{50} = 15 cm)	Wiremesh mattress d = 23 cm with stone fill Grade B (D_{50}=15 cm) on intermediate rubble layer (d = 25cm)	CC-blocks D_n = 30 cm hand-laid in single, diagonal lines	CC-blocks D_n = 30cm hand-laid in single, parallel lines	Interlocking CC-slabs (ship-lap type)	Wiremesh mattress d = 36 cm with brick fill	Interlocking CC-slabs (tongue-groove type) on intermediate layer	Rip-rap Grade C (D_{50} = 20 cm) Top 20 cm with stone pitching (d = 50 cm)	Rip-rap Grade C (D_{50} = 20cm) Top 20 cm with stone pitching (d = 40 cm)	As H-2 / Rip rap E + F 80 cm on 20cm
River side	Launching Apron at and below berm level (+14.5m to +15.3m PWD)		Dumped CC-blocks D_n = 30 cm	Dumped CC-blocks D_n = 35 cm	Dumped CC-blocks Edge us: D_n = 50 cm Center D_n = 35 cm Edge ds: D_n = 40 cm	Articulated Reno-mattress d = 23/36 cm; stone fill grade B, C, D (D_{50} = 25 cm) with inter-connecting steel wire ropes and anchor piles at berm level	Articulated CC-block mattress with inter-connecting steel wire ropes and anchor pipes at berm level	FORESHORE-mattress (collapsible block mattress with cement grout fill)	PROFIX-mattress (tubular fabric mattress with sand and sand-bitumen fill)	INCOMAT-sandflex mattress (collapsible block mattress with sand fill)	Rip-rap Grade F (D_n = 25-35-45cm)	CC-blocks D_n = 30 cm D_n = 35 cm (mixed)	
River side	Transition between launching apron and falling apron		CC-blocks D_n = 30cm	CC-blocks D_n = 30 cm	CC-blocks D_n = 35 cm				Rip-rap Grade E CC-blocks, D_n = 30cm *1	CC-blocks D_n = 35 cm *2			
River side	Falling Apron (level +14.5m PWD)	Dumped CC-blocks D_n = 30cm	Dumped CC-blocks D_n = 35 cm D_n = 40cm (mixed)	Rip-rap, Grade E (D_{50} = 30 cm)	Geo-sand-container Type C (180kg/No)	Geo-sand-container Type E (900 kg/No.)	CC-blocks D_n = 40 cm	CC-blocks D_n=40 cm / Geo-sand container D	CC-blocks D_n = 40/45 cm (mixed)	CC-blocks D_n = 35/40 cm (mixed)			
River side	Exposed edge of falling apron			Rip-rap, Grade F (D_n=25/35/45 cm)	Geo-sand-container Type D (250kg/No.)		CC-blocks D_n = 45 cm	CC-blocks D_n=45 cm / Geo-sand container E	Gabion sacks with stone fill Grade B (D_{50} = 15cm) (1,300kg/No)	CC-blocks D_n = 40 cm	Selected boulders D_n = 35-45 cm		

us = upstream
ds = downstream

*1 MIXED CC-BLOCKS 30cm + BOULDERS GRADE E IN EVELOPE OF DOUBLE LAYER CHAIN LINK FENCE

*2 MIXED CC-BLOCKS 35 cm + BOULDERS GRADE F IN EVELOPE OF DOUBLE LAYER CHAIN LINK FENCE

i.e. FEATURES INTERCHANGED C, D
CORRECTION AND ADDITION E TO H-2

REV.	DATE	NAME	DESCRIPTION	APPROVED
1	1.6.97	Anowar	AS BUILT DRAWING	

GOVERNMENT OF THE PEOPLES REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANISATION (WARPO)

BANK PROTECTION PILOT PROJECT FAP-21

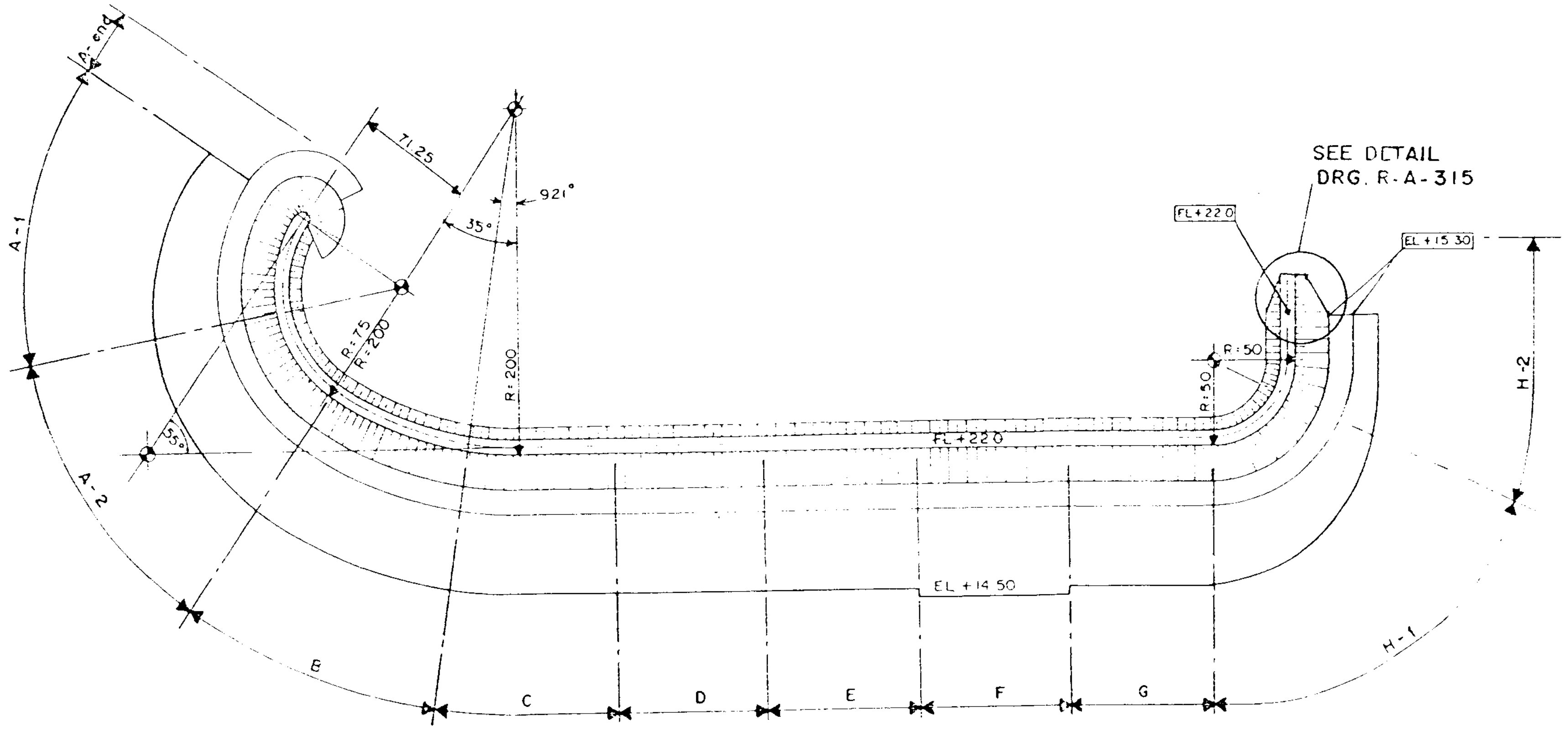
JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE
CONSULTING CONSORTIUM FAP 21/22
RHEIN-RUHR ING.-GES.MBH, DORTMUND/GERMANY
COMPAGNIE NATIONALE DU RHONE, LYON/FRANCE
PROF.DR. LACKNER & PARTNERS, BREMEN/GERMANY
DELFT HYDRAULICS, DELFT/NETHERLANDS
in association with
BANGLADESH ENGINEERING & TECHNOLOGICAL SERVICES LTD. (BETS)
DESH UPODESH LIMITED (DUL)

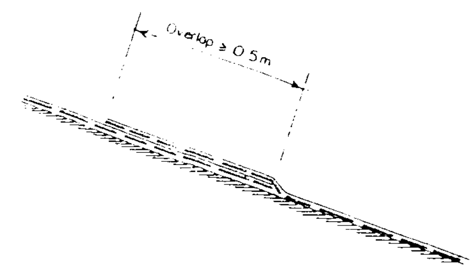
TEST SITE II - BAHADURABAD

DESIGNATION OF DESIGN SECTIONS AND MATERIALS

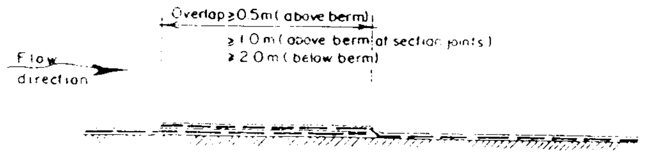
	NAME	DATE	SCALE 1 2000
DRAWN	ANOWAR	15-10-96	
CHECKED		18-10-96	FIG. 2.3 — REVISION 1
APPROVED			

DRAWING PHOTOREDUCED BY 50%





POSITION OF OVERLAPS DOWN-THE-SLOPE



POSITION OF OVERLAPS IN DIRECTION OF FLOW

NOTES

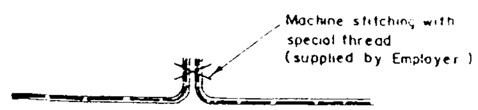
1. Levels refer to ±0.00 m PWD
2. Measurements are shown in meter
3. F.L. = Finished level
   E.L. = Excavation level
4. Geotextile filler materials and fabric mattresses will be supplied by the Employer in accordance with Specifications, Sub sections 220 and 230
5. Reference Drawings
   R-A-302 Detailed Layout of Test Structures
   R-A-303 Designation of Design Sections and Materials

SCALE 1 : 2000

i.e. FEATURES INTERCHANGED C,D

Test Structure			A-end	A-1	A-2	B	C		D	E		F	G	H-1	H-2	H-2 (end)	
Land-sided slope			GF-1	In all sections Geo-jute Soil Saver						Detail on Top of Embankment Drg. No. R-A-314						Village Road	
River Side	Approximate length along toe of upper slope (at berm level)		~ 8740	~ 74 70	~ 7470	~ 99 10	~ 93 20		88.0	90.0		88 0	100.0	~ 8275	~ 9760	+ 20.0-0	~30.0
	Geotextile filter mats above berm level	Spec Type	GF-1/-5	GF-1	GF-5	GF-2	Filter III on filter II	Khoa on filter II	GF-2	GF-1	GF-5	GF-1	GF-1	GF-4	GF-4/-2	As	GF-2
		Brand Name	BIDIM b7 HaTe O2214	BIDIM b 7	HaTe O 2214	BIDIM S 550			BIDIM S 550	DATEX AD1300	Hate 3 9014	BIDIM S 390	DATEX AD 1300	BIDIM S 700	HaTe E650/K251	H - 2	HateE650
	Geotextile filter mats at and below berm level	Spec Type	GF-1/-5	GF-2	GF-2	GF-4	GF-2		GF-4	FORESHORE-mattres (collapsible fabric block mattress with cement grout fill)		PROFIX-mattress (tubular fabric mattress with sand and sand-bitumen fill)	GF-1 (sub-layer to INCOMAT-sand flex mattress)	GF-1	GF-1	—	—
		Brand Name	BIDIM b7 HaTe O2214	BIDIM S 550	BIDIM S 550	HaTe K 251	DATEX AD1600		BIDIM S 700				BIDIM b 7	BIDIM S 390	BIDIM S 390	—	—

NOTE: In Section A and H-1 all geotextile filter mats are to be placed as single sheets in all other sections two sheets each are to be joined by stitching (prayer seam) to "twin sheets"



PRAYER SEAM
(without scale)

DRAWING PHOTOREDUCED BY 50%

REV.	DATE	NAME	DESCRIPTION	APPROVED
1	1.6.97	Anowar	AS BUILT DRAWING	

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MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANISATION (WARPO)

BANK PROTECTION PILOT PROJECT FAP-21

JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE

TEST SITE II - BAHADURABAD

GEOTEXTILE FILTER MATS
GENERAL ARRANGEMENT

	NAME	DATE
DRAWN	ANOWAR	17-10-96
CHECKED		19-10-96
APPROVED		

SCALE 1 : 2000

FIG. 2.4

REVISION 1

3 MONITORING OF THE TEST STRUCTURE

3.1 GENERAL

Since the final objective of this pilot project is to develop and optimize design criteria, cost-effective construction and maintenance methods which will serve as future standards appropriate for the prevailing conditions at the Jamuna and other rivers of Bangladesh, regular monitoring of the structures after their completion till end of the project in 1999 is one of the focal points of this pilot project.

Monitoring of the works undertaken at the test sites shall help to
- detect damages at an early stage;
- understand failing mechanisms, and
- plan suitable adaptation / repair works.

However, monitoring does not only refer to detecting damages on the structure but to observe their behaviour under load and to relate the loads to the structure's response. This requires on the one hand to monitor the loads (especially flow velocities, wave action etc.) and on the other hand to adapt the design rules. After adapting the design rules and the design, the works are to be adapted accordingly. Hence, the requirements of monitoring are to take care of the structures features as well as on the loads and natural effects which may influence the structures. Records are therefore to be taken of
- the natural conditions acting at the structures (water level rise and fall, waves, currents, precipitation, wind etc.);
- the morphological changes of the river in the area of the test structures;
- the movements of structures and important structural parts;
- the deterioration of materials used;
- the variations of surrounding river bed and bankline, and
- any damage by human and / or animal action.

Thereby it is of utmost importance for drawing right conclusions to record the above information with respect to
- exact location (referred to fixed points established in the hinterland);
- exact time of occurrence / survey;
- method of recording and equipment used;
- staff involved, and
- special observations etc.

All observations and data are entered in a Logbook developed for this particular purpose which, at the same time, serve as a check list for completeness of monitoring. Besides the results of regular hydrographic surveys, the Logbook is a basis for evaluation and selection of necessary measures to be taken. The Logbook and associated records enable to keep a continuous record of events showing the development of failure mechanisms and interrelation with acting forces.

Apart from daily routine observations, regular and periodic inspection programmes are carried out for each and every subject. However, time intervals have to be shortened in case deterioration is expected to increase not in line with the expectations and linear but at an accelerated pace. Additional inspections are required after extraordinary loading conditions, accidents etc.
The monitoring activities are subdivided into two main categories:

ɣ8

- survey of the properties and the behaviour of the structures, and
- reference measurements of physical phenomena that produce the loads on the structure.

The survey carried out under the monitoring programme is as follows:

- Logbook of activities and daily observations;
- Priority / alert information to FAP 21, Dhaka;
- Bathymetric surveys and recording;
- Water level recordings;
- Wind and wave recordings;
- Current measurements;
- Flow direction measurements;
- Topographic measurements;
- Bankline surveys;
- Changes/movements in the area of revetment, launching apron and falling apron of the individual sections;
- Meteorological measurements;
- Visual wave observations;
- Site processing quality control of data;
- Data transfer to FAP 21, Dhaka, and
- Detailed damage surveys during / after the flood period.

Final quality control of site data, final processing, presentation and evaluation is done in the office in Dhaka. The tasks are the following:

- Quality control of field monitoring;
- Final processing / presentation of survey results;
- Determination / confirmation of priority / alert situation;
- Initiation of emergency measures;
- Statistical evaluation of wind / wave records;
- Evaluation of water level recordings;
- Evaluation of current measurements;
- Comparison of results with applied design criteria, and
- Comprehensive annual report on results of field monitoring.

The organization of the monitoring activities and adaptation of works is similar to and comparable with that one of Test Site I which has been explained in more detail in the "Report on Monitoring and Adaptation at Kamarjani Test Site" of September 1996.

3.2 MONITORNG DURING THE MONSOON PERIOD 1997

3.2.1 <u>Preliminary Remarks</u>

Monitoring of the Revetment Test Structure stated already during the construction phase in January 1997 following the programme described in Section 3.1. Summaries of all activities have been reported in monthly monitoring reports. The progress of the whole project including the main results and observations of monitoring is reported in quarterly progress reports. In 1997 progress reports No. 15 to 18 have been published.

3.2.2 Bathymetry

Bathymetry surveys were mainly done to record riverbed changes in front of the test structure and to detect their influence on the stability of the structure, in particular to find out the behaviour/functioning of the falling aprons and launching aprons, since this is decisive for the overall stability of the test structure. The activities during the months of June to December 1997 are shown in Table 3.1.

The results of the main survey from June to December are presented in Annex C and some differential models in Annex D.

In addition to the main survey and the site survey bathymetric surveys were carried at each section daily during the monsoon. In the post monsoon season the period of time between the measurements had been increased. The results of these measurements are given as cross-sections in Annex F.

3.2.3 Topographic Measurements

The topographic measurements were done by using **E**lectronic **D**istance **M**easurement (EDM) equipment and levelling instrument. During the period from June to December the following works were performed:

06/04 - 07/04	BM pillar installation and Polygone survey
08/04	bankline survey and fix point description
15/05 - 18/05	topographic survey
27/05	bankline survey
17/06	topographic survey
24/06	BM installation
28/06	Zero value check
23/07	bankline survey from Bahadurabad Ghat to 500 m d/s from the test structure
23/07	boundary of deposition in Section B
15/08	bankline survey from Bahadurabad Ghat to 1.2 km d/s from the test structure
28/08	bankline survey from Bahadurabad Ghat to 1.2 km d/s from the test structure
17/09	bankline survey
29/10	BTM position of the water level gauges at Section B and Section H
31/10	bankline survey from railway ghat to 2 km downstream from Section H.

3.2.4 Measurement of Flow Velocity and Direction

Float track measurements were continued as well as measurements with the Valeport currentmeter. Results are presented in the monthly report on monitoring of the test structures. For details see also Annex E.

Date	Survey Area						
	June 1997	July 1997	August 1997	September 1997	October 1997	November 1997	December 1997
01			site survey	site survey			
02							
03							
04					main survey		
05					main survey		
06			main survey		main survey		
07			main survey				
08						main survey	
09						main survey	
10							
11							
12	main survey	site survey					
13				main survey			
14							
15			site survey				
16							
17							main survey
18							main survey
19		main survey					
20	main survey	main survey	main survey		main survey		
21		main survey	main survey		main survey		
22		main survey					
23							
24							
25				site survey			
26							
27							
28	main survey						
29	main survey						
30	main survey						
31							

Table 3.1: **Bathymetric survey at Bahadurabad Test Site from June to December 1997**

In addition, current measurements were carried out by a team of "Labor für Wasserbau" of Hochschule Bremen, Germany with drifter buoys using DGPS from beginning of June to August 22, 1997. Results were presented in the report on "Current Measurements at Kamarjani, Katlamari and Bahadurabad, August 1997" published in January 1998.

3.2.5 Observations

After the water level had started to rise in March the peak of the first half of the year was recorded at 18.56 m+PWD on June 19. The highest water level of the monsoon season 1997 of 19.69 m+PWD was measured on July 14. This value was 9 cm higher than that of 1996 and 79 cm less than in 1995. The following peaks were recorded at 18.56 m+PWD on August 16 and at 19.14 m+PWD on September 25. The last quarter of the year was characterized by a permanent fall of the water level with a total drop of 5.13 m and a maximum of about 3.40 m in October only. At the end of December 1997 the water level was at 13.64 m+PWD.

The following observations have been recorded for the period June to December 1997:

01/07 - 17/07	bank erosion downstream from the test structure
05/07 - 13/07	sliding down of cc-blocks in the area of falling aprons in Sections D to G
17/07	sedimentation on launching apron in Section B
20/07	strong wind up to 9 m/s
21/07	strong wind up to 9 m/s
31/07	bank erosion downstream from the test structure
02/08 - 31/08	bank erosion downstream from the test structure
04/08	strong wind up to 10 m/s
06/08	strong wind up to 10 m/s
10/08 - 11/08	eddies and return currents downstream from Section H
12/08	mean flow velocity over launching apron of Section E/F 2.1 m/s
28/08 - 29/08	erosion at launching apron of Section H-1
29/08	heavy rainfall (53 mm within 3.5 hours)
19/09	heavy rainfall (76.4 mm)
20/09	heavy rainfall (51.5 mm)
25/09	maximum water depth 33 m in front of Section G and H
27/09	wind speed 12 to 13 m/s
01/09 - 30/09	bank erosion from 100 m to 2 km downstream from Section H
04/10 - 17/10	slow bank erosion from 1.0 km to 2.5 km downstream from Section H
14/10	development of a char in front of the Railway Ghat
04/12 - 11/12	sliding down of cc-blocks in the area of falling aprons in Section F
25/12	slow bank erosion from 800 m to 1.0 km downstream from Section H.

The events during the monsoon 1997 and the post monsoon season can be summarised as follows:
During the construction phase the structure was protected by a natural earth dam. This was important for the completion of the falling aprons. The excavation level in the area of falling aprons was 14.50 m+PWD. During the first peak of the water level of 14.85 m+PWD end of March these areas were still

protected by the earth dam which, however, was permanently eroded. Assisted by relative low water levels in April, the earth dam remained intact for a longer period than expected. Its erosion increased with rising water levels in May and was finally washed away in the last week of June. Hence, the test structure became subjected to flow and wave attack and the falling aprons started to function. The Revetment Test Structure was strongly attacked by the river and also severe erosion downstream from the structure was observed. In July flow velocities up to 3.9 m/s were measured and a scour hole developed in front of Section D and E the deepest level of which was at -7.0 m+PWD. The river attack continued also in August and September just as the severe bank erosion downstream from the test structure which was about 100 m in September in an area about 1 km downstream from the structure.

Though the flow velocity decreased considerably up to about 1.6 m/s in September which was less than half of the velocity in July, the depth of the scour hole in front of the structure increased. The location shifted to Section G and H, where the deepest level was recorded at -10 m+PWD in August and at -14 m+PWD in September.

In October a mid channel char continued to grow in front of the Railway ferry ghat. This resulted in a reduction of width and depth of the channel near the bank upstream from the revetment test structure, whereas the western channel developed further and attacked the river bank just downstream from the structure. Considerable erosion of some 50 m were recorded in October and this process continued till end of the year.

No damage to the test structure was observed, the falling aprons functioned well as expected and at the end of the year sedimentation in the area of the falling aprons of Section G and H and the channel occurred.

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4 ANALYSIS OF THE MONITORING SURVEYS

4.1 INTRODUCTION

For a better understanding of the hydraulic load on the test structure (Section 4.3) first a description of the development of the bankline is given in Section 4.2. The observed behaviour of the revetment toplayers, launching aprons and falling aprons is explained in Sections 4.4 to 4.6 respectively. Various aspects of the development of the scour hole in front of the revetment structure are described and analysed in Section 4.7.

4.2 BANKLINE

The bank line started to erode in spring 1997 when the water level started to rise. On June 11 the cofferdam protecting the revetment structure still functioned, but after its breach on June 20 the test structure came under flow attack. During the first half of July strong bank erosion had been observed downstream from Section H. This erosion stopped on July 18 after the second flood peak. Between June 11 and July 23 the bank had been eroded over a distance of about 100 m at an average rate of about 2 m per day. Later it continued at a lower rate of 1 to 1.5 m/day up to September 17, then it stopped. The bank erosion shifted about one kilometre in downstream direction during September (see Fig. 4.1). Downstream from Section H an embayment developed during this monsoon more or less as assumed in the design of the test structure.

Upstream from the revetment structure no bank erosion occurred during the monsoon 1997. However, during the coming years erosion upstream from the structure might also result in an upstream embayment as assumed in the design of the test structure. At the end of the monsoon the downstream edge of the falling apron protruded about 80 m into the channel and the crest of the revetment in Sections G and H 20 m only.

4.3 HYDRAULIC LOAD

4.3.1 Introduction

The attack of the river on the revetment test structure is characterised by the following parameters:

- maximum water levels;
- maximum flow attack, and
- maximum wave attack.

The water levels have been measured on a daily basis throughout the whole year (Sub-Section 4.3.2) and the flow velocities in different ways during special surveys (Sub-Section 4.3.3). The wave height in the channel had been estimated only during storms and windy days (Sub-Section 4.3.4). The daily rainfall had been recorded during the whole monsoon.

4.3.2 Water Levels

The water level was measured at two staff gauges which were located at the upstream and downstream side of the revetment test structure. The upstream water level gauge had been shifted from Section A to Section B on August 27. The downstream gauge was placed in Section H. The reading of the water level at the staff gauges is scheduled normally at 8.00, 13.00 and 17.00 hours daily. In general, the readings from 8.00 hours were used as a daily averaged value presented in hydrographs (see Fig. 4.2). The accuracy of the reading is estimated at ± 0.01 m.

The hydrograph observed at the test site during the monsoon 1997 had 4 peaks above 18 m+PWD. However, the maximum water levels in the peak flows were all below the design water level of 21.10 m+PWD (25 years return period) or 21.40 m+PWD (100 years return period) (see Table 4.1).

Peak flow	Date	Time reading	Upstream gauge	Downstream gauge	Estimated return period
		hours	m+PWD	m+PWD	Years
1	June 19	8.00	18.56	-	< 2
2	July 14	8.00	19.69	19.55	< 2
3	August 16	8.00	18.56	18.48	< 2
4	Sept. 25	8.00, 13.00	19.14	19.05	< 2

Table 4.1: Maximum peak water levels

The duration curve of the water levels shows that the water levels were lower in 1997 than in the previous two years (see Fig. 4.3). In 1995 the water level gauge had probably been placed more upstream from the location in 1997. The low water levels in December and January are the same as in 1996. At 17.3 m+PWD the lower part of the flood plain started to be inundated, the upper part of the flood plain had a level of about 18.5 m+PWD.

The water level gradient is the difference in the upstream and the downstream water level divided by the distance between the staff gauges along a flow line which is about 550 m. The water level gradients during the monsoon have been presented as function of the water level in Fig. 4.4. This graph shows that the water level gradient was higher before the peak on July 14 than after this peak for the same water level. This tendency is clearly shown because that peak flow had a single peak. However, it is less clear for other peakflows because these flows had multiple peaks. The flood wave had a steep front followed by the recession limb with lower gradients.

During high flow the Chezy formula reads:

$$U = C\sqrt{h.i}$$

in which

U	=	flow velocity (m/s)
C	=	Chezy coefficient ($m^{0.5}/s$)
h	=	water depth (m), and
i	=	water level slope (-)

In the graph of Fig. 4.5 the flow velocity has not been measured at the same time as the water level readings along the flow line between the staff gauges, therefore the relationship is not accurate and the values of the Chezy coefficients are probably too low. However, this graph explains that the high flow velocities occurred before the flood peak and as soon as the water level recedes the flow velocities reduce strongly. This has been observed several times by the monitoring team.

29

4.3.3 Flow Velocities and Flow Pattern

Flow velocities have been measured by Valeport velocity meter in the vicinity of the test structure only on September 11. They were rather low that day, see the depth averaged flow velocity in Fig. 4.6. The standard deviation of the slowly accelerating flow along the edge of the falling apron varied from 0.06 to 0.09 $\bar{u}$ in Sections B to G and 0.2 to 0.3 $\bar{u}$ at the downstream termination in Section H, where flow separation occurs ($\bar{u}$ = the local depth averaged flow velocity).

However, flow velocities have been determined more frequently by float tracking. Measurements were carried out by the monitoring team of FAP 21 and by a team of Hochschule Bremen, see special report on their measurements (Consulting Consortium FAP 21, January 1998). Above the launching apron the flow velocities have been measured by float tracking almost daily.

Maximum flow velocities

Special float tracking by Hochschule Bremen showed maximum flow velocities on August 12 over the launching aprons increasing in downstream direction (see Table 4.2). The measurements on July 13 and August 16 have been carried out by Hochschule Bremen, those on July 14 and August 12 by FAP 21. These measurements have been selected because they have measured the highest flow velocities. The tendency that the flow velocity increased over the launching apron has also been observed by the monitoring team, which had measured maximum flow velocities on August 12 when the water level raised for the third peak in the hydrograph. The monitoring team used a water bottle as float which had been thrown into the water at Section B. Benchmarks were the reference points at the centre of Sections B, C, D, E, F and G. An operator recorded the time, when the floating bottle passed the benchmarks. The results have been presented in the Logbooks.

Section	Flow velocity (m/s)			
	July 13	July 14	August 12	August 16
C	1.0	0.64	-	-
D	1.2	0.7	1.18	0.88
E	1.5	0.8	1.78	1.28
F	1.7	1.2	2.10	1.35
G	1.8	1.4	1.97	1.46
H	-	1.6	-	-

Table 4.2: Maximum flow velocities measured above the launching aprons

Downstream from the test structure a maximum flow velocity of 3.9 m/s has been measured at the surface above the scour hole near the downstream termination. The maximum flow velocities above the falling apron were measured on July 13 (see Table 4.3). This survey was made just one day before the flood had reached its highest water level in Bahadurabad, see also hydrograph in Fig. 4.2. The flow velocities in Table 4.3 vary because of the location where the measurements were made varied. Above the falling apron a sharp gradient in flow velocities exists and the floats are sensitive to turbulent action as eddies and vortices. The general tendency is that the flow velocity in the approach channel increased as soon as the flow line bent due to the structure. Along the edge of the falling apron the flow velocity gradually increased and the maximum flow velocities have been measured above the downstream scour hole, see example of the float trackings in Fig. 4.7.

Section	Flow velocity (m/s)			
	Surface	-3m	-6m	depth averaged and smoothed
C	2	-	2.6	2.4
D	1.8	-	2.2	2.5
E	2.7	-	2.4	2.7
F	2.8	3.1	2.6	2.9
G	3.1	3.4	3.1	3.1

Table 4.3: Maximum flow velocities measured above the falling apron on July 13, 1997

In the additional model investigation in 1993 maximum flow velocities of 4.4 to 5.7 m/s had been measured with a return period of 25 year. Compared with the maximum velocities in Table 4.3 these flow velocities are too high, as already mentioned in the report of this model investigation. However, part of these differences are probably because of the difference in the approach channel in the model (where parallel flow occurred) and in Bahadurabad (where the approach channel made an angle of about 13 degrees with the revetment structure).

Horizontal profiles

Bathymetric surveys and float trackings have been used to make a model of the flow velocity distribution in front of the revetment, (see Fig. 4.8 to 4.13). The bankline had been selected as boundary where the flow velocity is zero. A float track is sensitive to turbulence and eddies in the river flow. Therefore the flow velocities determined by float trackings show some scatter. These three models are based on only a few float trackings and therefore the model shows some irregularities. It is recommended to use more float trackings close to the revetment structure in new surveys.

In general the flow accelerated flowing along the revetment structure. Therefore the highest flow velocities have been observed near the downstream end of the structure. Cross-section G has been chosen to present the flow velocities (see Fig. 4.9, 4.11 and 4.13). The advantage of the model is that it smoothes the flow velocity distribution (see Fig. 4.8, 4.10 and 4.12).

During the design phase it had been planned to dredge the underwater slope of the revetment structure and a horizontal flow velocity distribution had been assumed. Later, however, it was decided to construct a falling and launching apron just above standard low water level. Following the original design a smooth straight sloping surface had been assumed and that the flow velocity above the deepest point at about 120 m from the crest of the revetment is 100 %. When, however, the falling and launching apron started to operate and its material has partly fallen, the actual surface deviated from the assumed one. Therefore, this comparison is not straight forward, but the flow velocity distribution along Section G on September 08 follows very good the assumed distribution in the initial situation without bank erosion as considered in the design phase (see Fig. 4.11). In the other figures the flow velocity distribution deviates slightly because of the shape of the underwater surface.

The maximum measured flow velocity above the deepest point of the scour hole is 2.7 m/s which is below the design flow velocity of 3.5 m/s.

4.3.4 Wind and Waves

The wind speed, wind direction and wave heights have been recorded in the logbooks. During the monsoon the wind direction varies between east and south. Six stormy days with a maximum wind speed of 8 to 10 m/s have been observed, but the maximum recorded values are at 12 to 13 m/s (see Fig. 4.14). The wind direction was away from the test structure and therefore waves did probably not break on it.

In the logbook of August it is mentioned that the wave heights are the maximum observed wave heights in the channel, often observed about 100 m away from the falling apron. A maximum wave height of 1 m has been estimated on July 03, when the wind direction was east (see Fig. 4.14). The second highest wave height had been estimated at of 0.7 m. These observations are probably not very systematic and accurate, because the wave height can vary quickly during a day.

A wave runup of about 1 m along the revetment slope has been measured on September 27.

4.4 TOPLAYERS AND FILTERLAYERS

The monitoring team reported some repair in Section E on September 22, where cuts had been observed and bulging of the ship-lap interlocking concrete slabs near the water level because of accumulation of migrated sand at about 2 to 3 m below the water level. The depths of the cuts were 0.15 to 0.2 m (see sketch in Fig. 4.15).

The site report mentioned heavy rainfall in Bahadurabad on June 15 as well as on September 19 and 20 (65 and 51 mm rainfall respectively). After this rainfall some settlements have been observed in the upper revetment in Sections E and F above the water level line (see sketches in Fig. 4.15). The ship-lap interlocking slabs have a variable shape regarding roundness of their corners, thickness of the slabs, and variable gabs between the slabs.

The hypothesis is that rainfall entered through the gaps between the blocks and ran down, transporting some of the subsoil. Just below the water level the subsoil material has been deposited. Bulging of the surface has been observed at several places. The contact between the blocks and the geotextile might have been low on several places because of 'beam action' between several blocks.

These cuts have been observed in the filter test investigation in 1994 due to simulated wave action or rainfall and a critical gradient has been determined (Consulting Consortium, 1994). The flow phenomena near the water level are similar due to rain or wave rundown. However, the forces induced by wave rundown are probably much higher than the forces induced by the rain water flow over and in a revetment. During the monsoon no significant wave heights have been observed breaking on the revetment.

In subsoil A which is sandy silt type ML to CL pipes develop until a complete failure. Subsoil B which is silty sand type MS is characterised as a very dangerous soil because of relatively low permeability, relatively high compressibility and low shear strength. In the filter test investigation segregation of soil fractions and/or piping have been observed as the main failure mechanism. These pipes develop under the geotextile.

The flow along the revetment slope had a gradient of about 0.3. The results of the filter test investigation show that a small and un-equal distributed weight from the toplayer on the geotextile reduces the critical gradient from 0.5 to 0.2 in case of subsoil A (see Table 4.4). Beam action between the concrete slabs has probably reduced locally the weight pressure by the concrete slabs on the geotextile to zero. This might have reduced the critical gradient below the actual gradient. This explains the observed damage in Section E.

Construction type	Critical gradient (-)	
	Subsoil A	Subsoil B
cc-blocks on geotextile	0.5 to 0.6	0.9 to 1.25
boulders on geotextile	0.2	0.6 to 0.7

Table 4.4: Results of filter test investigation

It had been observed that the cc-blocks in Section D were stable. The submerged weight of these cc-blocks with D_n = 30 cm is 300 kg/m^2. Without beam action the weight of these blocks is equally distributed over the geotextile and the critical gradient is about 0.5 which is higher than the actual gradient. Therefore, no damage has been observed in that section.

4.5 LAUNCHING APRON

The flow velocities above the launching apron had been analysed in Section 4.3.3. It can be concluded that the launching aprons can resist a flow velocity of 2 m/s in the flow over the aprons without any observed damage. This has been expected, because the design flow velocity is 3.5 m/s.

The edge of the articulated Reno mattresses in Section C had been launched early in the monsoon season (see Photographs 3 and 4 in Annex M of Progress Report 19, Consulting Consortium, January to March 1998), and on July 11 an erosion of 1.5 m had been observed.. The mattresses filled with boulders did fall well, but due to sharp bending the wiremesh of some few cages broke up and some boulders fell out.

Therefore it is recommended to use stronger wiremesh.

4.6 FALLING APRON

4.6.1 Monitoring Data

The monitoring team had observed the erosion process in different sections daily and recorded in the logbook (see Table 4.5). The cofferdam initially protecting the revetment structure still functioned on June 11 but on June 20 the test structure came under flow attack. It seems that some erosion had already occurred in almost all sections along the edge of the falling apron before June 24.

Standard cross-sections have been selected in the middle of each section. These cross-sections have been surveyed almost daily, from the actual bank line to about 200 m in the channel. For a very detailed comparison of cross-sections the accuracy of the positioning system is a limitation. A check has been made on August 20 and the standard deviation of the positioning by DGPS (Trimble) was about 4 m and by EDM (Astech) about 2 m (see Appendix E of the Monthly Report, Consulting Consortium FAP 21/22, August 1997).

A few soundings have been made parallel to the crest of the revetment, but it was difficult to sail the same straight line in subsequent surveys. Therefore it is difficult to compare different longitudinal soundings with each other. It was decided to omit these soundings in the standard survey programme.

In addition to the visual inspection of the monitoring team photographs were taken regularly. A selection of these photographs have been included in the Progress Reports.

Date	Section	Material	Observation
24-26/07	B	geo-sand container type D	erosion over 4 to 5 m.
27/07	B	geo-sand container type D	erosion
before 24/06	C	geo-sand containers type E	erosion over 2 to 7 m. Materials fallen on the new bed
24-26/06	C	geo-sand containers type E	erosion over 2 to 7 m
26/06 & 02/07	C	geo-sand containers type E	bed scour
01-02/07	C	geo-sand containers type E	some erosion
08-09/07	C	geo-sand containers type E	erosion occurs at slope
10-11/07	C	geo-sand containers type E	erosion at river bed
15-16/07	C	geo-sand containers type E	erosion
26/07	C	geo-sand containers type E	siltation
27/07	C	geo-sand containers type E	erosion
31/07	C	geo-sand containers type E	siltation
before 24/06	D	0.45 m cc-blocks	blocks have been fallen up to 5 m from the edge
before 24/06	D	0.40 m cc-blocks	blocks have been fallen up to 1.5 m from edge
24-26/06	D	0.45 m cc-blocks	erosion over 5 m
24-26/06	D	0.40 m cc-blocks	erosion over 1.5 m
26/06 & 02/07	D	cc-blocks	bed scour
03-04/07	D	cc-blocks	erosion occurs along slope
05/07	D	cc-blocks	cc-blocks fallen over slope
08-09/07	D	cc-blocks	erosion occurs at slope
10/07	D	cc-blocks	cc-blocks fallen on slope
11/07	D	cc-blocks	cc-blocks fallen on river bed
12/07	D	cc-blocks	erosion
15/07	D	cc-blocks	erosion over 2 m
16/07	D	cc-blocks	erosion along slope protection
26/07	D	cc-blocks	siltation
29/07	D	cc-blocks	erosion
29/08	D	cc-blocks	erosion over 2 m
before 24/06	E	geo-sand containers type E	containers have been eroded over 7 m
before 24/06	E	geo-sand containers type D	small erosion of containers
before 24/06	E	0.45 m cc-blocks	exposed edge has been eroded over 7 m
24-26/06	E	geo-sand containers type E	erosion over 7 m
24-26/06	E	0.45 m cc-blocks	erosion over 7 m
01-02/07	E	cc-blocks	bed scour
03-04/07	E	cc-blocks	erosion

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Date	Section	Material	Observation
05/07	E	cc-blocks	cc-blocks fallen over slope
08-09/07	E	cc-blocks	bed scour
10/07	E	cc-blocks	erosion over 3 m
11/07	E	cc-blocks	cc-blocks fallen on river bed
13-16/07	E	cc-blocks	erosion
26/07	E	cc-blocks	1 m siltation along slope
27/07	E	cc-blocks	erosion
before 24/06	F	gabion sacks	erosion over 5 m from edge
24-26/06	F	gabion sacks	erosion over 5 m
(3, 4, 6, 11, 12, 15, 16-17)/07	F	gabion sacks	erosion
07/07	F	gabion sacks	scour
13-14/07	F	cc-blocks	cc-blocks fallen on falling apron
28/08	F	cc-blocks	erosion over 3 m DAL 78 to 100 *)
before 24/06	G	0.45 m cc-blocks	erosion over 4 m from edge
24-26/06	G	0.45 m cc-blocks	erosion over 4m
(1-10, 12-14, 15-16, 29)/07	G	cc-blocks	erosion of cc-blocks
11/07	G	cc-blocks	2 m scour at river bed
28/08	G	-	material from sec-F deposited in sec. G
28/08	H	rip-rap grade C	top part between DAL 58 to DAL 67 gone
29/08	H	rip-rap grade C	erosion between DAL 50 to DAL 70

*) DAL = distance along line

Table 4.5: Observation of the erosion processes in the sections during the 1997 flood

4.6.2 <u>Slopes</u>

(i) Slopes undisturbed by a structure:
Natural slopes which are not influenced by a nearby structure have been observed during the erosion of the dam in front of the falling apron during spring 1997. The upper part of cross-sections C, E and G surveyed on March 06, April 04 and May 06, 1997 at the time that the dam in front of the falling apron eroded had an average slope of 1V:1.95H and a steepest slope of 1V: 1.7H.

(ii) Section C:
The geo-containers of the falling apron in Section C did not fall gentle, as can be seen from photographs. A steep, almost vertical bank or bluff had developed around SLW. The closely packed containers fell one by one from the edge. The coverage on the underwater slope has not been measured.

(iii) Section H:
From photographs it can be seen that the boulders of the falling apron in Section H-1 had fallen and the geotextile BIDIM S 390 hanging on a steep slope. The falling apron had been eroded over a distance of about 25 m after the flood season. This erosion started when a scour hole had developed near the downstream termination (see Fig. 4.16).

Already in 1903 Spring (Spring, 1903) reported that he had observed the formation of the steep sub-surface sand cliffs along banks. He realised that a vertical cliff as measured in the falling apron in various test sections weakens the protection of the embankment. The thickness of the falling apron should be sufficient to cope with these cliffs. According to his field experience it might be possible to prevent the development of such a cliff by an apron of basalt stones which have a high specific density. He suggested that the slope of the falling apron should be not steeper than 1V:2H, to prevent slipping of the apron stones above the low water level. If the apron stones slip, a bare patch sandy or clay bank without any protection may be eroded in only a few hours. The steep slope 1V:1.6H in the lower part of the facing slope of Sections C and H might be considered as a factor contributing to the risk of the small slides or slips as observed in these sections.

Varma et al (1989) mentioned that observations at guide banks on various rivers have shown that the actual slopes of launched aprons range from 1V:1.5H to 1V:3H and in some cases flatter. However, in most cases the average approximates 1V:2H. This is confirmed by the falling aprons in Bahadurabad. Varma mentioned: "that physical model tests have shown that an apron does not launch satisfactorily unless the angle of repose of the underlying material is flatter than that of the protective work. This means that with an apron laid on the river bed consisting of alternate layers of sand and clay, stones slide down as sand layers scour and clay layers subside, causing uneven cliffs so that the apron cannot launch uniformly. Stones fall to the bottom and are washed away." This is confirmed by the falling apron in the Sections C and H-1.

The comparison with guidelines from literature, where it is recommended to assume a slope of 1V:2H for the slope of the falling apron, shows that in Bahadurabad slightly steeper slopes have developed on average: 1V:1.6 H. This slope is steeper than the natural slope of a free eroding bank, 1V: 1.95H, because of the protection by cc-blocks, boulders, rip-rap and gabions.

(iv) **Small slides or slips:**

The regular falling process has been disturbed by at least two small slides in Sections C and H (see Table 4.6). The slopes of the falling apron in those Sections C and H have been surveyed in June, August and September:

upper part: the average slope is 1 V:1.0 H and a steepest slope of 1 V: 0.5 H, and
lower part: the average slope is 1 V:1.6 H and a steepest slope of 1 V: 1.2 H.

The steepest slopes in the upper part of the cross-section occurred at the shortest distance to the scour hole (see Fig. 4.16), where the cross-sections G_T and H-1/x intersect the deepest part of the scour hole. The upper part had a thickness of 4.5 to 5.5 m from 9 m+PWD to the excavation level of the falling apron at 14.5 m+PWD. Apparently, these slides are confined to this cohesive toplayer and therefore they are limited in size and the risk of larger slides is probably small.

In the period of the second slide from August 28 to 30, the water level dropped 0.1 to 0.15 m/day. However, during the first slide of geo-containers in Section C the water level raised 0.2 to 0.3 m/day.

Material of apron	Cross-section Number	survey before slide date	survey after slide Date	visual observation of erosion dates	minimum level m+PWD
Geo-sand container	C	June 11	June 20	beginning July	10
Rip-rap grading	H-1	August 20	August 27	August 28-29	9

Table 4.6: Two small slides in falling apron process

Comparison with model investigations:

- These slides have not been observed in the physical model investigation in France, where the falling apron of cemented cubes had been placed on a subsoil of sand. These slides are important because the material of the falling apron is mixed up with the subsoil during a slide. This means that no perfect coverage of the surface can be obtained after such a slide has occurred. And this weakens the protection of the under water slope of the test structure. In general repair measures are necessary to maintain a good protection of this under water slope. In the winter season 1997/1998 such a repair measure has been carried out in the area of the downstream termination, where the deep scour hole was close to this falling apron
- In the physical model investigation in Faridpur a pre-shaped falling apron had been made of cement layer with a smooth surface. The hydraulic roughness of that layer is much lower than the hydraulic roughness of the falling apron. The flow pattern over the falling apron had been influenced by this difference in surface roughness. Therefore it is recommended to represent the hydraulic roughness of the falling apron on scale in the model.

The subsoil investigation during the Study Phase of this project included boring BA-1 in the Bahadurabad area (see Annex 13 of the report of the Study Phase, Consulting Consortium, June 1993). This boring shows that half of the 6 m thick toplayer is of medium dense, brownish grey sand with silt and a trace of mica and the other half of this toplayer consists of low plastic, grey silt (80 % silt and 20 % sand) and a trace of clay. Under this toplayer the soil consists of medium to very dense sand (80 % sand and 20 % silt). This toplayer is more cohesive and probably more erosion resistant than the sublayers.

The shear strength of the soil consists of two components, cohesion and friction between the particles:

$$\tau = c + \sigma \tan \varphi$$

in which

c	=	cohesion (kN/m^2)
φ	=	angle of shear resistance (angle of internal friction, degrees)
σ	=	normal stress (kN/m^2), and
τ	=	shear stress (kN/m^2)

The characteristic values of these parameters in Bahadurabad are mentioned in Table 4.7.

This subsoil investigation in Bahadurabad shows that this toplayer has cohesive properties and that below this toplayer the subsoil consists of non-cohesive medium to dense sand. Therefore steeper slopes can develop in that toplayer. The thickness of the toplayer depends on the location. However, along the Jamuna River a cohesive toplayer on non-cohesive sandy layers is common, according to FAP 1 surveys (Halcrow, 1993). This means that falling aprons with a slope of 1V:0.5H in a toplayer and 1V:1.6H in sublayers might be expected as a general shape.

	c (kN/m^2)	φ (degrees)
toplayer	0 to 11.7	29 to 35
sublayers	0	33

Table 4.7: Soil parameters in Bahadurabad

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4.6.3 Erosion Velocity

The falling process progressed very gradually, for example in Section F (see Table 4.8). The erosion rate started with about 0.2 m/day in June/July and it slowed down in July and August. In September sedimentation occurred on the facing slope. The average bed level of the channel in front of Section F eroded in the period June to August, but it sedimented in September. This sedimentation on the facing slope of the falling apron is part of the overall sedimentation in the channel. Gradual erosion of the falling apron in Section F has been analysed to determine when the falling apron became stable after falling and could resist parallel flow attack, maximum 2.5 to 3 m/s at a depth of 3 to 6 m below the water surface according to the measurements made by the Hochschule Bremen.

The conclusion is that the facing slope of the falling apron had continued to erode at an average speed of 0.1 m/m^2/day as long as the channel eroded. The erosion only stopped due to the general sedimentation in the channel. This means that the sloping face will probably start to erode as soon as the channel will erode again. Consequently, the sloping face has not reached an equilibrium position during the moderate attack by a parallel flow. This might be because of the cc-blocks and the gabion sacks form a rather uniform material. To stabilise the facing slope of the falling apron a layer of graded material should be dumped to form a natural filter layer.

Period			Distance from crest revetment (m)		
start	end	days	70 - 80	80 - 90	90 - 100
June 26	July 05	9	0.21	0.23	0.05
July 05	August 01	36	0.11	0.09	0.09
August 01	August 29	64	0.07	0.13	0.11
August 29	October 01	97	-0.02	-0.01	0.001

negative value = sedimentation
positive value = erosion

Table 4.8: Average erosion/sedimentation velocity ($m/(m^2.day)$) in Section F

The slide with boulders and rip-rap grading in Section H-1 gradually extended in downstream direction and the scar disappeared. However, this did not happen with the slide in Section C with geo-containers and cc-blocks. The maximum erosion velocity during small slides which takes one or two days, has been determined from the cross-section surveys. The duration of a slide has been assumed at two days. Near the deep scour hole the maximum erosion velocity has been measured at Section H-1: 1.5 $m/(m^2.day)$. No monitoring data is available to assess the maximum erosion velocity in the slide in Section C.

4.6.4 Coverage of Falling Area

Surveys with side-scan sonar and sub-bottom profiler were used to determine the coverage of the surface of the falling apron. Separate reports have been prepared on these surveys.

The thickness of the falling apron was at the edge 6 to 7 times the size of a cc-block in Section G. These cc-blocks had fallen nicely with a full coverage of the bluff or steep edge. However, Section F with a thickness of only 3 B (B = block size) at the edge showed an unprotected strip of the steep edge over a height of about 1 m.

A kink in the depth measured in cross-section surveys indicates a certain coverage down to this kink, (see example in Fig. 4.18). This bend has been caused by the slowly eroding surface formed of the falling elements. Also Spring (1903) mentioned this phenomena from field observations on the large rivers in the Indian sub-continent (see his sketch in Fig. 4.19).

A comparison with the results of the physical model investigation in France shows that the upper slope of the falling apron did not have a good coverage, because it was rather steep. In that investigation it was not possible to reduce this upper slope and to improve its coverage. This tendency was also observed in sections with a thin falling apron of cc-blocks or boulders near the downstream termination in Bahadurabad.

4.6.5 Design Formula

The design formula for the volume of the apron per running meter is (see Consulting Consortium January 1995, Special Report No. 4, Falling apron investigations):

$$V = c_1 . y_s . B$$

in which

V	=	volume of the apron (m^3/m')
c_1	=	emperical coefficient, 4 to 5 (-)
y_s	=	scour depth (m), and
B	=	block size or stone size (m)

The width of the falling apron is 1.5 y_s and the thickness is 2.7 to 3.3 B.

During the design of the falling apron it was considered to increase this volume by 15 % to allow for the loss of stones or blocks in slides. Because these slides have not been observed in the physical model investigation, which results had been used to determine this formula.

The design level is - 20 m+PWD and the maximum observed scour depth is -14 m+PWD (-27.5 m SLW). Therefore the falling apron had not been tested to its full extend.

In principle the following consequences might be considered:

- A 2 B thick falling apron needs repair measures after the falling process had started, to obtain a complete coverage of the toplayer in critical situations;
- If the subsoil is erosion resistant then it might be considered to delay these repair measures in non-critical situations, and
- A 5 to 7 B thick falling apron layer does not need repair measures and it will provide full coverage during the falling process.

Future monitoring of this test structure will be used to support these tentative conclusions. Further, it is recommended to increase the thickness of the falling apron in the design formula, when this apron is designed on a cohesive toplayer of silts (group CL and ML), to allow for some loss of apron material during the mixing process of apron material and the subsoil during a slip.

4.7 SCOURING

4.7.1 Introduction

In front of the revetment test structure a deep scour hole developed during the 1997 monsoon. The scouring started as the approach flow was deflected by the test structure at the beginning of the monsoon. Initially two scour holes developed, one in front of the centre of the revetment structure and a separate one near the downstream termination of the structure because of strong turbulences in a vortex street associated by a flow separation between an eddy and the main flow. This vortex-street caused a deep scour hole and bank erosion downstream from the test structure. However, upstream of the structure bank erosion did not occur during the 1997 flood and therefore a scour hole near the upstream termination did not develop. During the monsoon the revetment structure became more exposed to flow attack due to this bank erosion downstream from it. This resulted in coalescing scour holes into one scour hole due to protrusion scour and flow separation at the end of the monsoon. However, during the receding limb the scour holes tended to develop again separately.

The monitoring data of this scour hole has been analysed. The most relevant aspects have been described in the following sections.

4.7.2 Maximum Scour Depths

The scour depth is defined as the distance between the deepest point of a scour hole and the upstream bed level in the approach channel. The upstream bed level is defined as the average level of the deepest area of the channel and this area is about 30 to 40 % of the total channel area. The channel bed is almost constant during the monsoon, it varies from -2.5 to +2.5 m+PWD. The bed levels and the scour depths, which have been surveyed during the monsoon 1997, have been presented in Fig. 4.20.

The depth of the scour hole gradually increased from 2 m in May to 14 m end of September at an average rate of 0.1 m/day. This erosion rate is similar to the falling rate of the falling apron in Section F (see Sub-Section 4.6.3). This gradually increasing scour depth is different from the quick development of the local scour holes near Kamarjani groynes in the 1995 monsoon: 0.25 m/day near groyne G-1, 0.3 m/day in the scour hole near G-2 and 0.4 m/day in the scour hole near G-3. An erosion rate of 0.3 m/day or more is one factor which increases the risk of large slides as observed near G-2 and G-3. Fortunately, the slow erosion rate of the scour hole near the revetment structure has reduced considerably the risk of large slides damaging this new test structure.

In the physical model investigation executed during the Planning Study Phase in 1992 (Consulting Consortium, June 1993) the scour hole had been studied near the upstream termination. In the additional model investigation in 1993 (Consulting Consortium, August 1994) the complete revetment had been tested and also the scour hole near the downstream termination. The average bed level was at -13 m SLW in the model. This is the same as the bed level in the field where it varied in the approach channel between -11 and -15 m SLW.

In the model the maximum scour depth near the downstream termination was 10 m with a 25 year return period (see test T14 where protrusion = 50 m, radius = 50 m and side slope = 1:2). In Bahadurabad (protrusion = 80 m, radius = 50 m, side slope varied from 1:2 to 1:1.6) the maximum scour depth was 14 m.

The prefixed cemented falling apron slope in the model was too smooth compared with the falling face in the field. In the model where the flow was more or less parallel to the revetment structure, the aprons had launched completely, but in Bahadurabad the approach flow made an angle of about 13 degrees with the revetment structure where only a small edge had fallen. The oblique flow attack, the extra protrusion and the steeper slopes explain that in the field about 4 m deeper scour depth was measured than in the model.

The conclusion is that the maximum scour depths in Bahadurabad test site can most probably reasonably well be reproduced in a physical model. However, in the previous physical model investigation not the most unfavourable flow attack had been tested by an approach channel which makes an angle with the revetment structure. The design channel determined for the model investigation had the same cross-section as observed in the field. The applied statistical method to determine the design cross-section as an envelope curve of surveyed cross-sections seems to be a good method.

Based on the results of the physical model investigations the design scour depth has been assumed 12 to 14 m near the upstream termination and 10 to 12 m near the downstream termination. A design scour depth of 6 m had been assumed between these two scour holes (Procurement and construction report, Test Site II, April 1995, see Fig. 4.21). The monitoring of scouring during the monsoon 1997 shows that mainly due to an oblique flow attack instead of parallel flow which had been tested in the physical models deeper scour holes can develop than designed. The upstream scour hole did not develop, because no upstream bank erosion had occurred. In front of Section E which had been designed in between the two scour holes a maximum scour depth of 8 to 10 m had been measured, which is 2 to 4 m more than designed. Therefore, a modification of the design scour depth is proposed as presented in Fig. 4.21: a design scour depth of at least 12 to 14 m in front of the revetment. Near the upstream termination it might be necessary to increase the design scour depth if the protrusion will increase because of upstream bank erosion. In the design an upstream bank erosion had been anticipated of only 100 m. If the upstream bank erosion is more than 100 m then the fallen apron of the upstream termination will probably need reinforcement.

4.7.3 <u>Location of the Maximum Scour Depth</u>

The location of the deepest point of the scour hole has been determined from bathymetric maps. In some surveys the scour hole has an elongated flat bottom, which is indicated by two or more points connected by a line in Fig. 4.22. In general this deepest point shifted in downstream direction during the monsoon.

During June and July the scour hole developed at some distance from the falling apron with the deepest points in front of Sections D to F. When the scour hole became fully developed in August and September the deepest point of the scour hole shifted towards the falling apron in front of Sections G and H. The eddy with return current downstream from Section H started to develop during those months. In September the scour hole reached its maximum size. In October it started to silt up and the location of the deepest point did not move anymore.

In the physical model tests in 1993 and in 1995 completely fallen and launched aprons had been assumed. However, during the 1997 monsoon these aprons were fallen or launched only partly in Bahadurabad. The difference in protrusion and slopes explains that in the physical model the maximum scour depth was about 25 to 50 m more close to the downstream termination than measured in the field.

4.7.4 Slopes and Volume

The longitudinal slope of a scour hole is the slope parallel to the main flow direction and the transverse or side slopes are almost perpendicular to these longitudinal slopes. The longitudinal slopes of the scour have been determined from three surveys at the time the scour hole had been fully developed (see Fig. 4.8, 4.10 and 4.12 with bathymetric maps where also the longitudinal alignment of the Sections A-A', B-B' and C-C' respectively is shown). In October the scour hole started to silt up and its shape changed with sedimentation from upstream, the flat bottom became sloped 1V:40H. The average slopes have been determined (see Table 4.9). The fully developed scour hole is characterised by a long flat bottom with a length of 15 y_m, with y_m = maximum scour depth (m) and upstream and downstream slopes of about 1V:25H. The area A_l (m^2) of a longitudinal section of such a scour hole is $A_l = 40\ y_m^2$.

Date	Upstream slope (1V:xH)	Downstream slope (1V:xH)	Volume below -4 m PWD (m^3)
August 15	27	21	173,000
September 13-14	23	25	393,000
October	24	31	379,000

Table 4.9: Slopes along a longitudinal section in scour hole and its volume

End of August the sloping face of the falling apron had been developed to the deepest point of the scour hole. Therefore, the left side slope which has been covered by cc-blocks and boulders, was 1V:2H to 1V:3H. The right side slope of the fully developed scour hole was much more gentle, on average 1V:15H (see Table 4.10). Even more gentle slopes exist during developing stages of the scour hole. Approximate transversal area A_t (m^3/m') of a cross-section of the scour hole perpendicular to the main flow direction is about $A_t = 10\ y_m^2$.

The combination of these gentle longitudinal and steeper side slopes results in elongated scour holes as can be seen on almost all bathymetric maps. These slopes can be compared with those measured in confluence scour holes near Bahadurabad surveyed by the River Survey Project in 1993: longitudinal slopes of 1V:100 to 200H for the upstream slope and 1V: 250 to 400 H for the downstream slopes. They are less steep as slopes in the scour hole in front of the revetment structure. The side slopes of a bend scour hole south of Bahadurabad varied between 1V:15H and 1V:25H and this confirms the right side slope of the scour hole in front of the revetment structure (River Survey Project, Special Report 24, Morphological processes in the Jamuna River, October 1996).

Section	Date	left slope (1V:xH)	right slope (1V:xH)
H-1	August 30	1.5 to 2.3	12 to 19
F	August 29	7.5	11
D	Sept. 12	6	15
C	July 07	-	25

Table 4.10: Transverse or side slopes in the scour hole

The volume of the scour hole has been calculated from three bathymetric surveys (see Table 4.9). These volumes are the volume of the scour hole below -4 m+PWD. This an arbitrarily level just below the deepest part of the approach channel.

The total volume V_t of the scour hole can be estimated at $V_t = 200y_m^3$.

This formula is consistent with the formulas for the cross-section areas in longitudinal and transversal directions. The maximum volume of the scour hole is about 0.4 million m^3 (y_m= 13 m below the channel bed in the approach channel).

4.7.5 Calculation of the Maximum Scour Depth

Two calculation methods are described in this section, one for the time dependent development of the scour hole and the other one for the equilibrium scour depth.

(i) The Time Dependent Development of the Scour Hole

The scour hole in front of the revetment structure developed as a function of time during the monsoon of 1997. If the maximum depth of he scour hole $y_m(t)$ is smaller than the upstream water depth h_0 (m) then the following simple relationship can be applied (Hoffmans and Verheij, 1997):

$$\frac{y_m(t)}{h_0} = \left\{\frac{t}{t_1}\right\}^{\gamma}$$

in which

h_0	=	water depth upstream from scour hole (m)
y_m	=	maximum scour depth in scour hole (m)
t	=	time
coefficient γ (-)	=	0.7 to 0.8 for this type of structure with three dimensional scour holes

The time t_1 (hours) has been defined as the time at which the scour depth is equal to the upstream water depth:

$$t_1 = \frac{330.h_0.\Delta^{1.7}}{(\alpha.U - u_{cr})^{4.3}}$$

With $\Delta = 1.65$, $u_{cr} = 0.4$ to 0.5 m/s this formula reduces to:

$$t_1 = 773.h_0^2.(\alpha.u - u_{cr})^{-4.3}$$

in which α increases from 1.2 at the start of the monsoon to 2 at the end of the monsoon.

The volume of a section of scour hole parallel to the main flow direction is A_l (m^3/m):

$$A_l = c_a.y_m^2$$

in which c_a = coefficient (-) depending on the shape of the scour hole and y_m follows from the first formula in this section.

Sediment transport entering the scour hole from upstream reduces the scour hole depth. An approximate approach is:

$$A_{l,r} = A_l - c_b q_s t$$

In which $c_b q_s$ = part $_{cb}$ of the sediment transport which settles in a scour hole ($m^2/(m.s)$).

$$q_s = \{0.00017 h_0^{-0.2} + 0.15 * 0.000752\} u (u - 0.025 \ln(240 h_0))^{2.4}$$

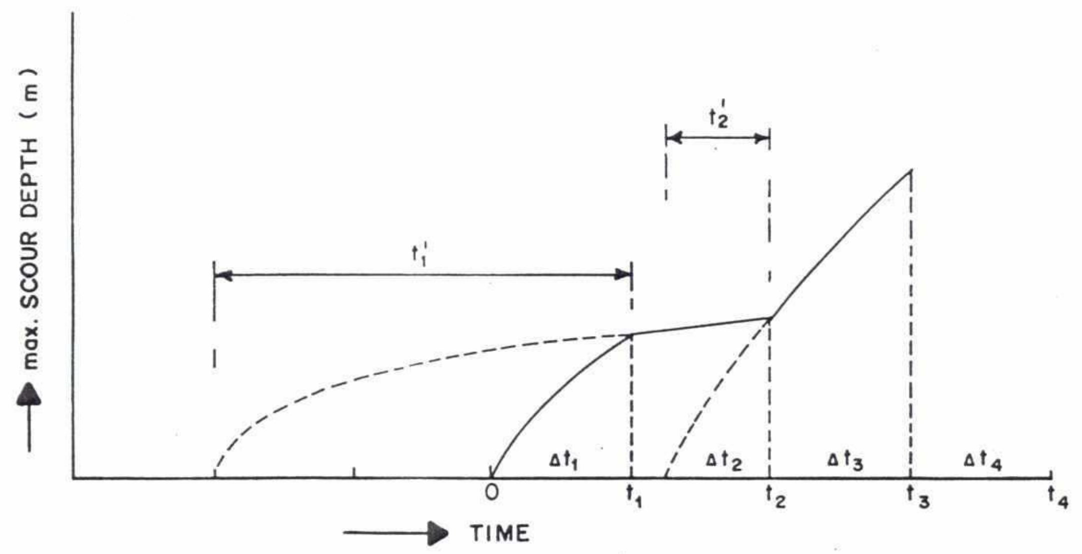
The reduced maximum scour depth follows from:

$$y_{m,r} = \sqrt{(A_{l,r} - c_b q_s t) / c_a}$$

In the Jamuna River the coefficient $c_b = 0.1$ because only about 10 % of the sediment transport (also called near bed sediment transport) will settle in a deep scour hole of (0.5 to 1) h_0 depth. If the scour hole is only a few meters deep (< 0.5 h_0), even less will deposit, and the reduction by incoming upstream sediment transport can be neglected. In this calculation method U, u_{cr} c_a and c_b are the main calibration parameters: u_{cr} = 0.4 to 0.5 m/s, c_a = 20 and c_b = 0.1 or less.

The flow velocity U has been estimated from float trackings and from a relationship with the local slope. It has been observed various times that after a flood peak the flow velocities reduce considerably. Then also the water level slope reduces. The relationship is shown in Fig. 4.4.

The results of the calibration of this calculation method on the maximum scour depth near the downstream termination are shown in Fig. 4.24, using the calculation method as indicated in the following sketch. The value of the parameter α.u is one of the most important parameters for the calibration. The parameter α increased from 1.2 in June to 2 in August and September. It is recommended to develop this method further with other calibrations. This will present a better knowledge of the values of these coefficients and their physical meaning.



Sketch: Principle of the calculation time dependent development of the maximum scour depth

(ii) **Equilibrium Scour Depth according to Ahmad**

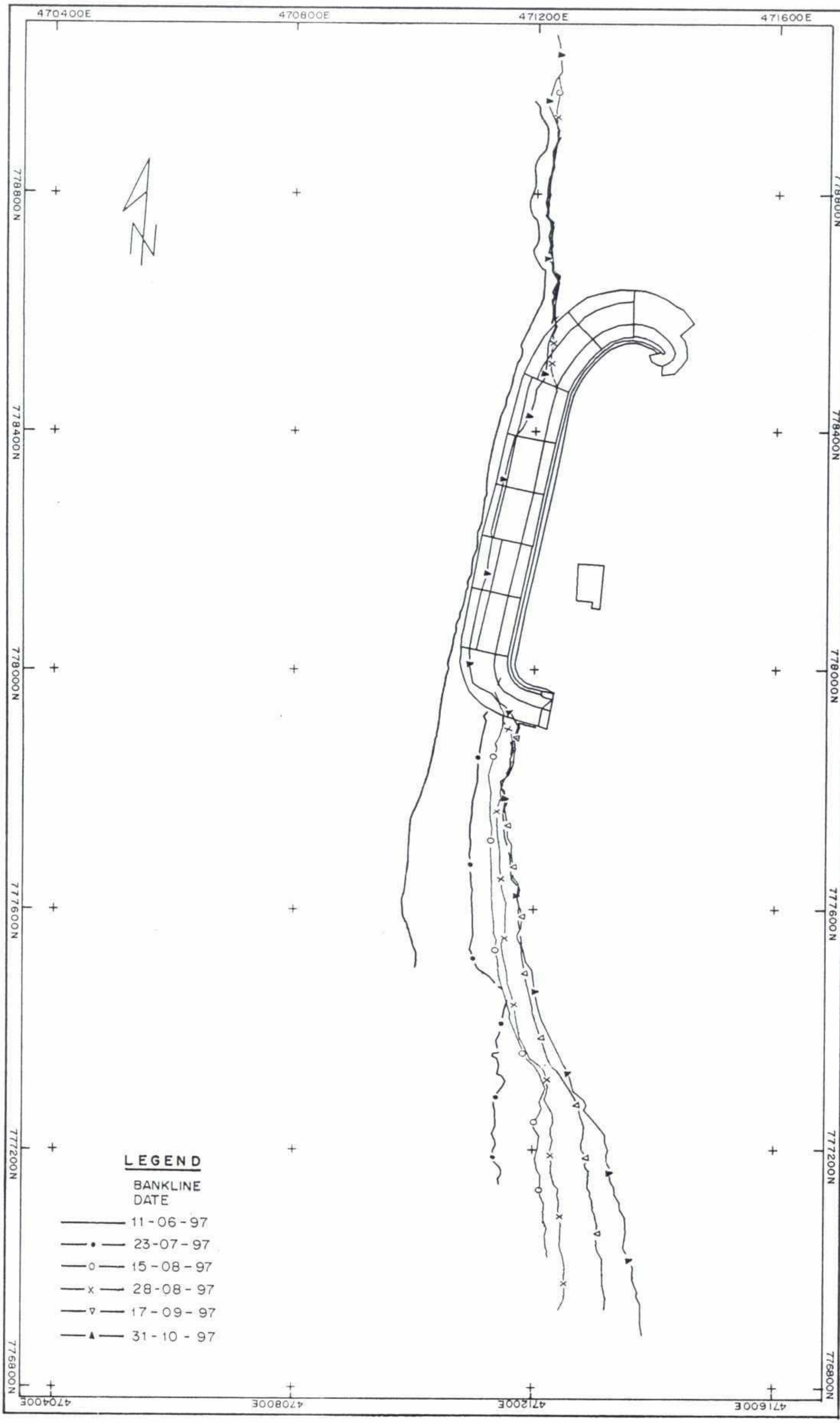
The formula for the calculation of the equilibrium scour depth has been published by Ahmad (1953):

$$y_{m,e} + h_0 = K\{h_0 \frac{B}{B-b} u\}^{2/3}$$

in which

K	=	coefficient ($m^{-1/3}s^{2/3}$)
B	=	flume width or channel width (m)
b	=	protrusion length of the revetment (m)
u	=	upstream flow velocity (m/s)

In the Jamuna river $\frac{B}{B-b}u$ is considered as the increased flow velocity near the structure. At the time the maximum scour depth had been reached end of August and September 1997 the coefficient was K = 2.5 to 3.5. The upper value is a safe first estimate for the design of this type of structures. In the initial stage in June and July the scour hole developed slowly and the observed scour depth was not the equilibrium depth. The value of coefficient K was low, between 1 and 2. In the final stage end of September and October the scour hole was too large for the actual flow conditions and started to sediment. In that stage large values of K = 4 to 5 have been calculated form the measurements.



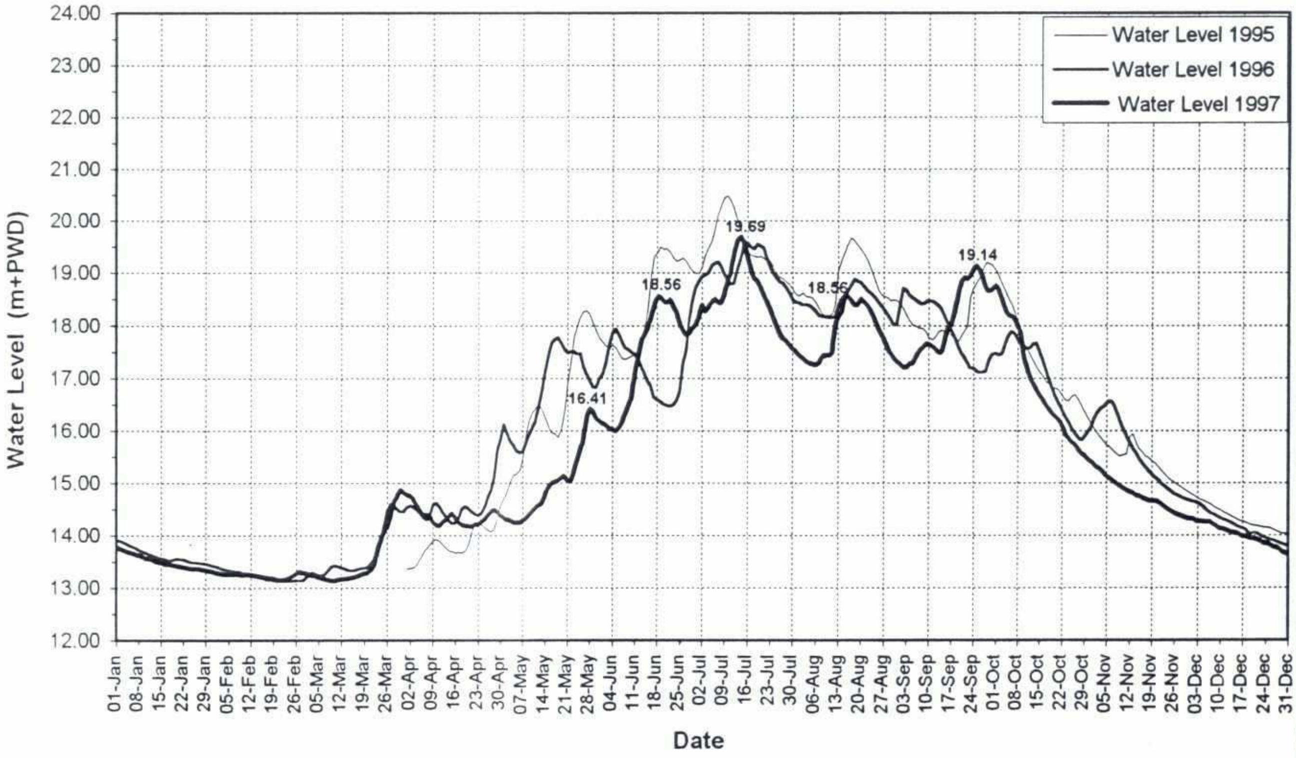
BAHADURABAD TEST SITE

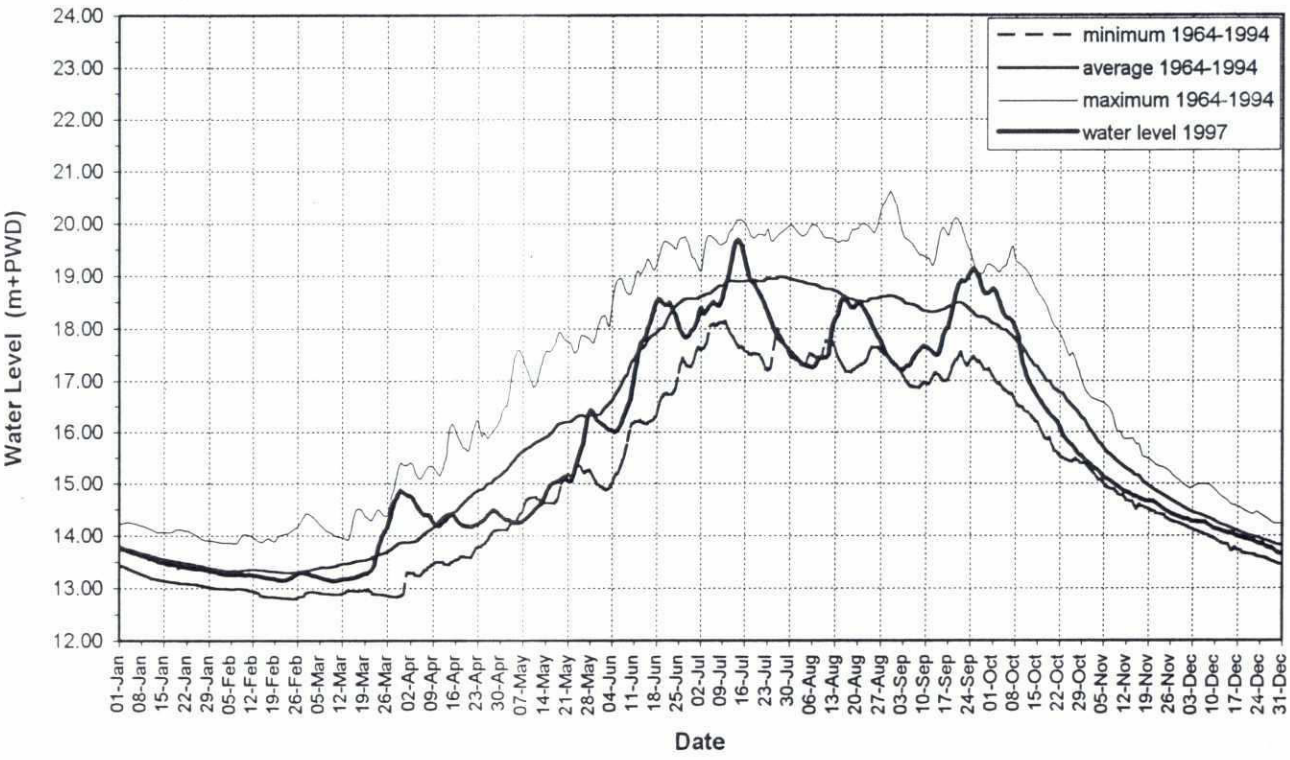
Bank erosion during monsoon 1997

FAP - 21

BANK PROTECTION PILOT PROJECT

FIG. 4.1





BAHADURABAD TEST SITE

Hydrograph in 1995, 1996 and 1997

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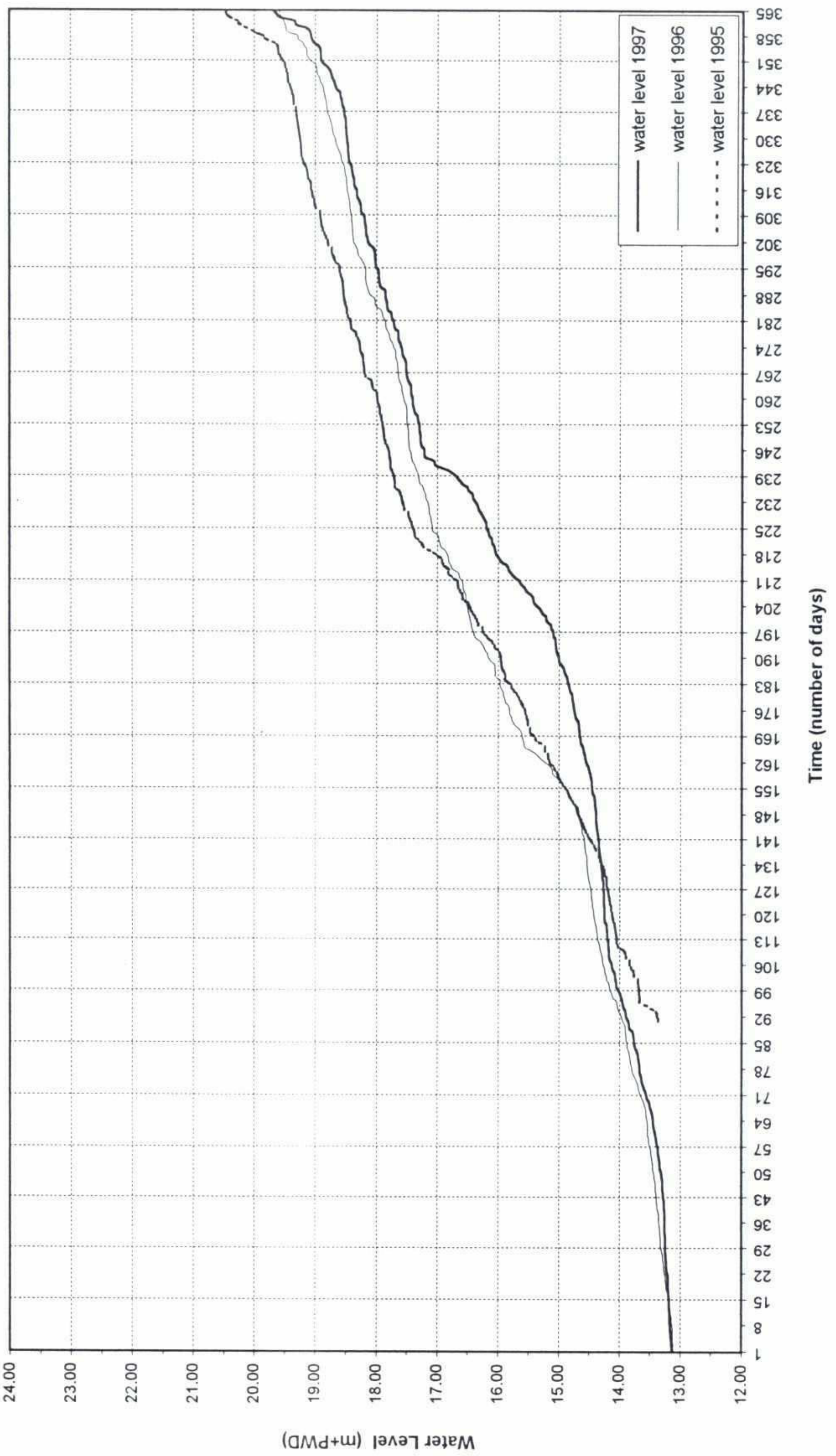
FIG. 4.2

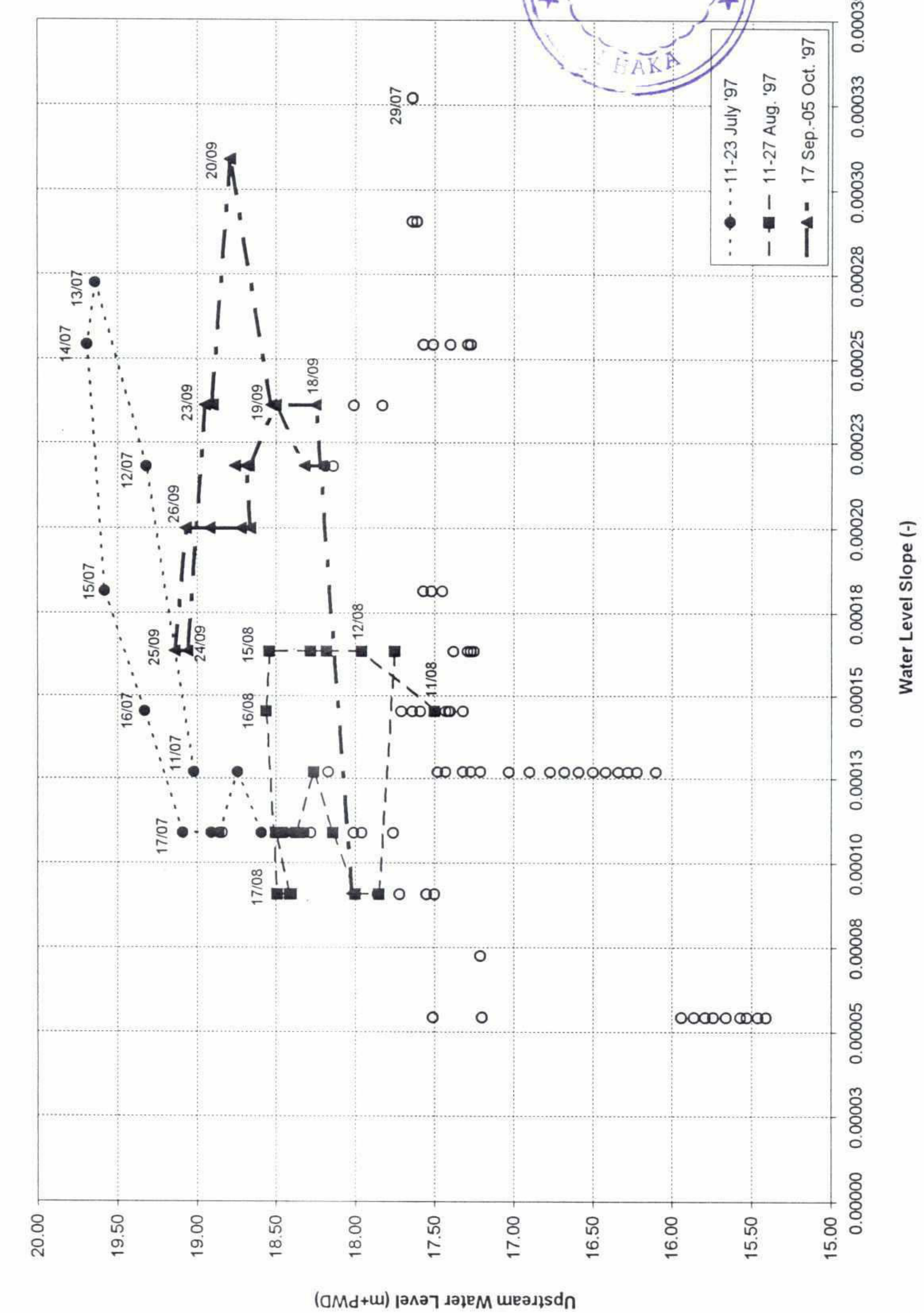


BAHADURABAD TEST SITE
Duration curve of water levels in 1995, 1996 and 1997

FAP – 21
BANK PROTECTION PILOT PROJECT

FIG. 4.3

80



BAHADURABAD TEST SITE
Water level slope as function of the water level in 1997

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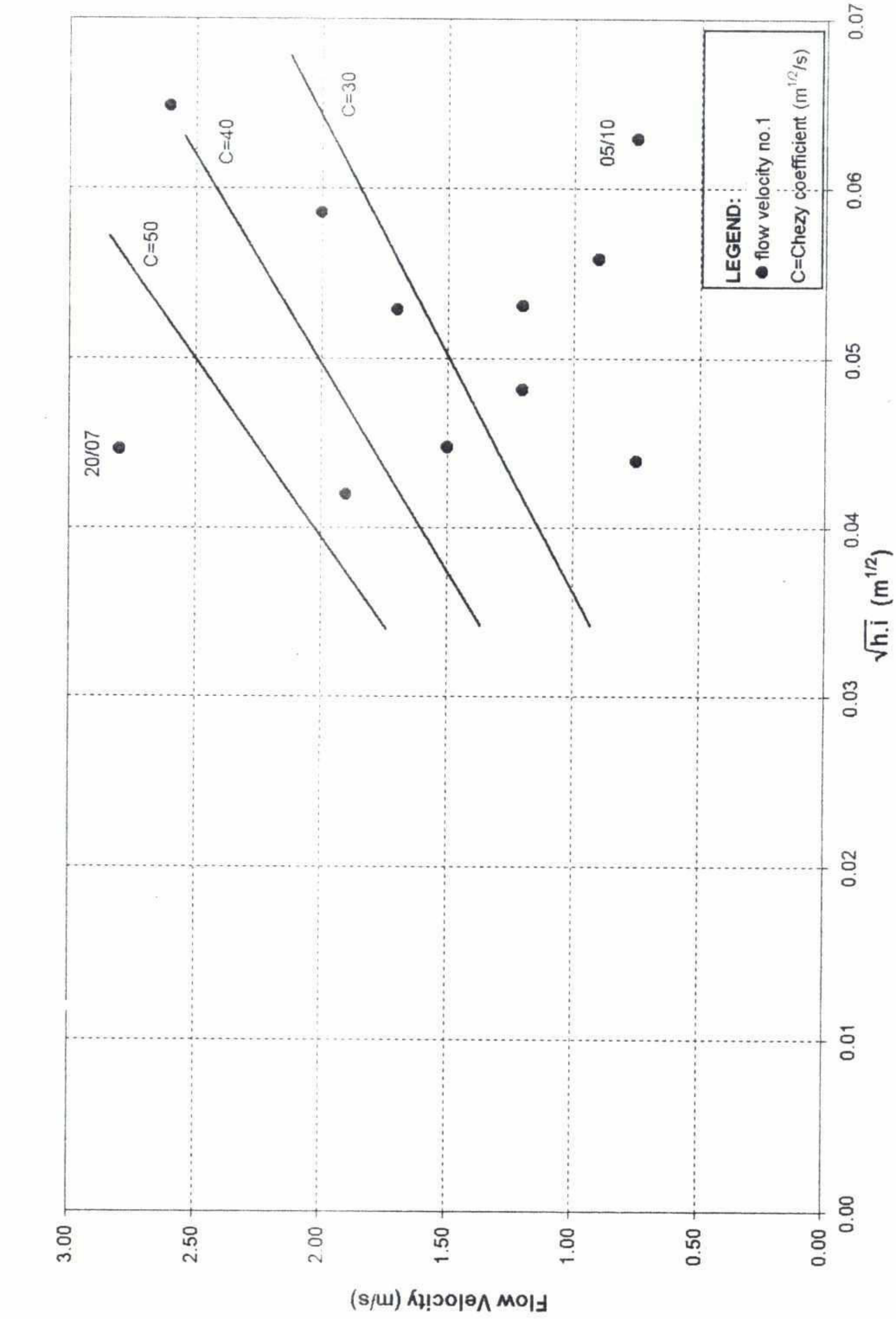
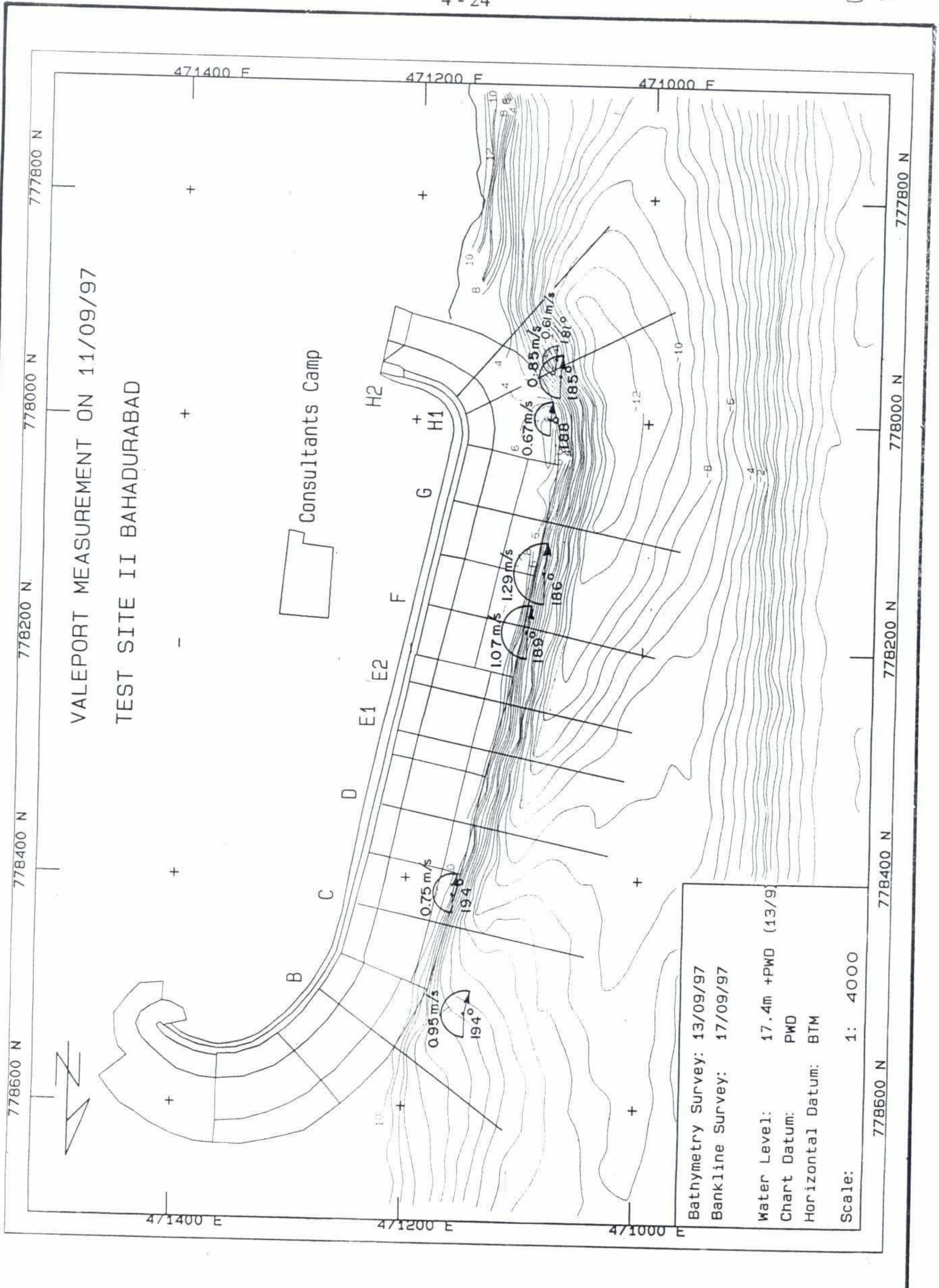
FIG. 4.4

82



BAHADURABAD TEST SITE	FAP – 21
Application of the Chezy formula	BANK PROTECTION PILOT PROJECT
	FIG. 4.5

8ↄ

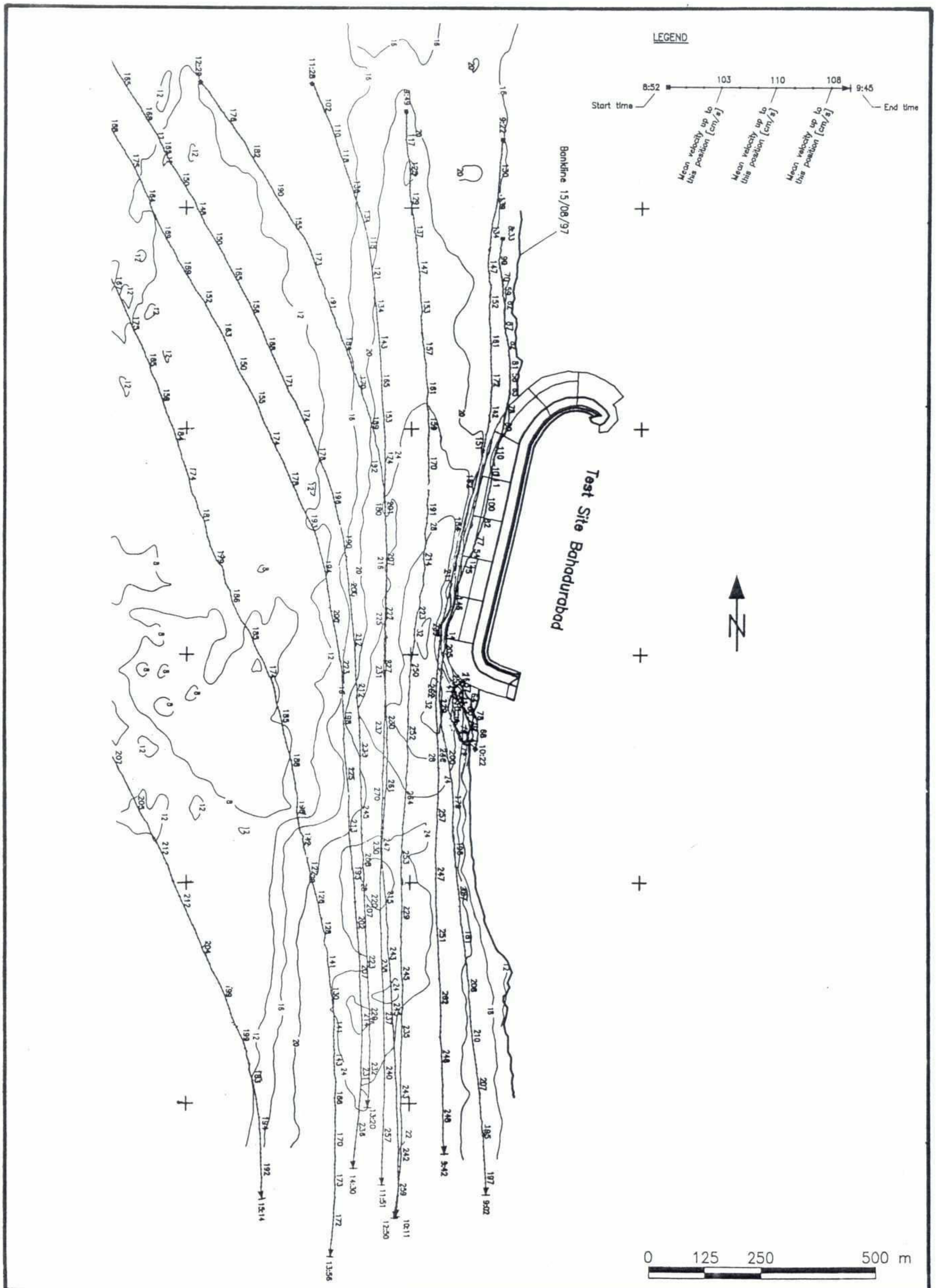


BAHADURABAD TEST SITE
Depth averaged flow velocities on September 11, 1997

FAP – 21
BANK PROTECTION PILOT PROJECT

FIG. 4.6

86



BAHADURABAD TEST SITE

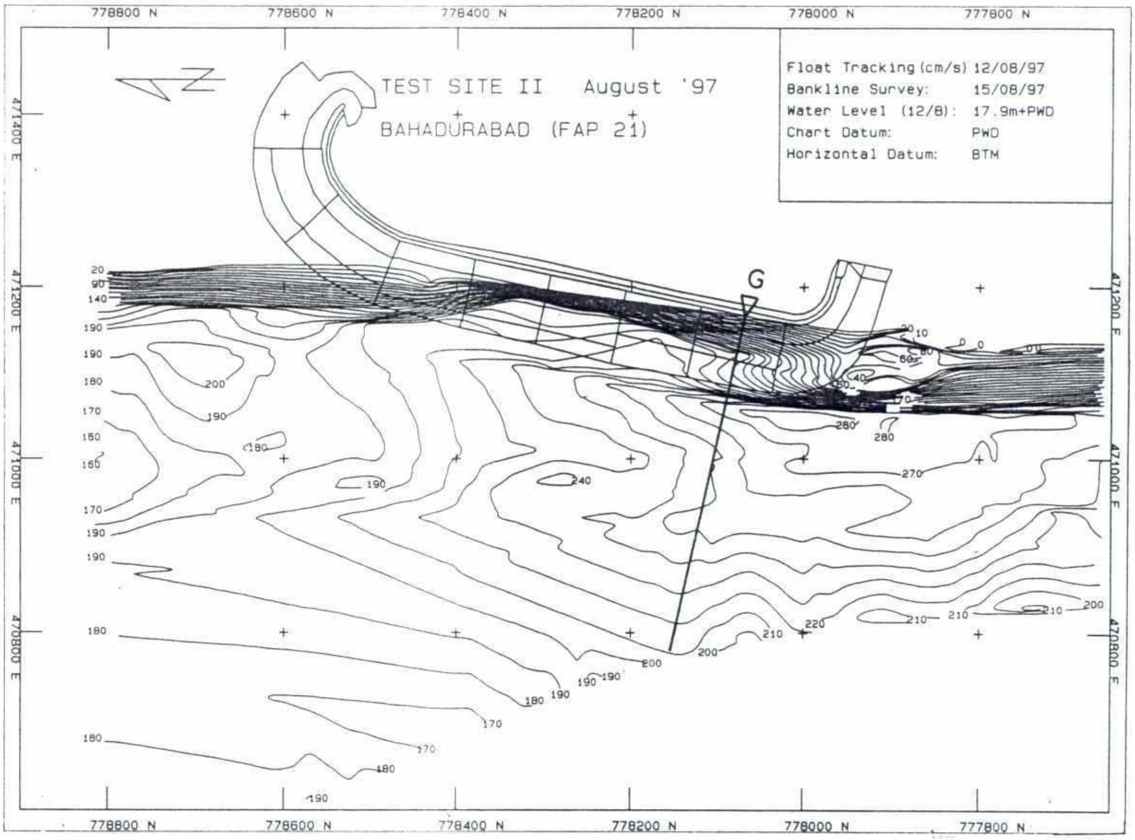
Float Tracking on August 12, 1997

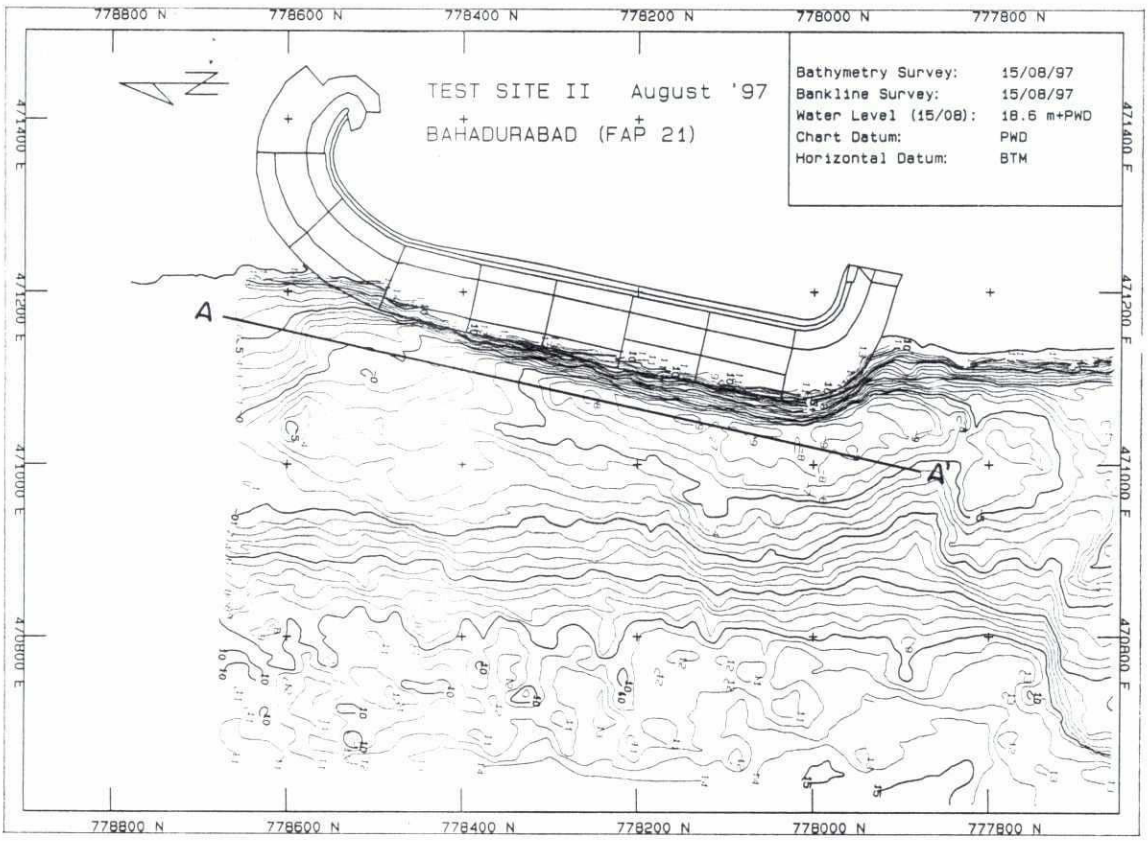
FAP – 21

BANK PROTECTION PILOT PROJECT

FIG. 4.7

88





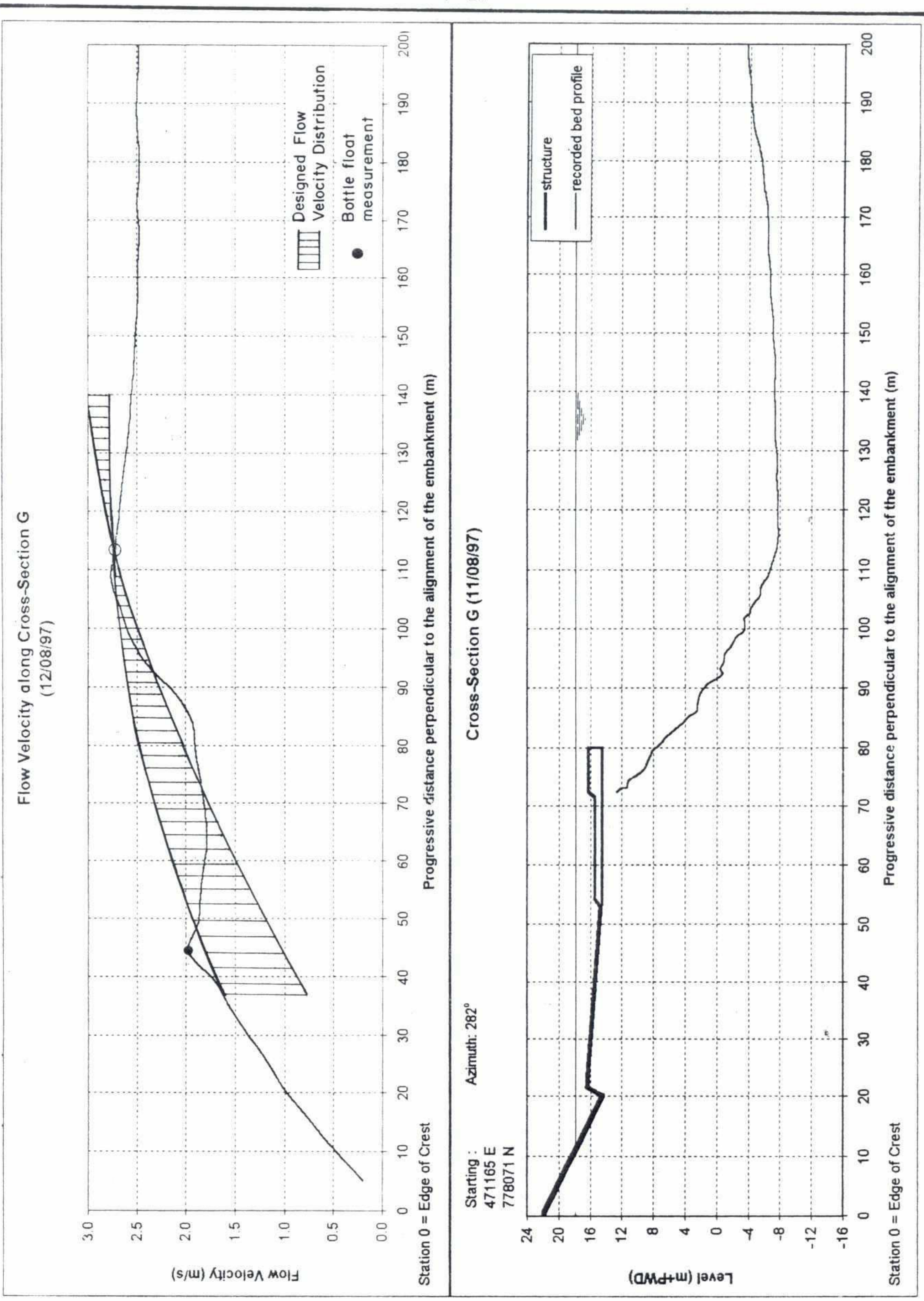
BAHADURABAD TEST SITE
Flow velocity model and bathymetry
August 1997

FAP - 21

BANK PROTECTION PILOT PROJECT

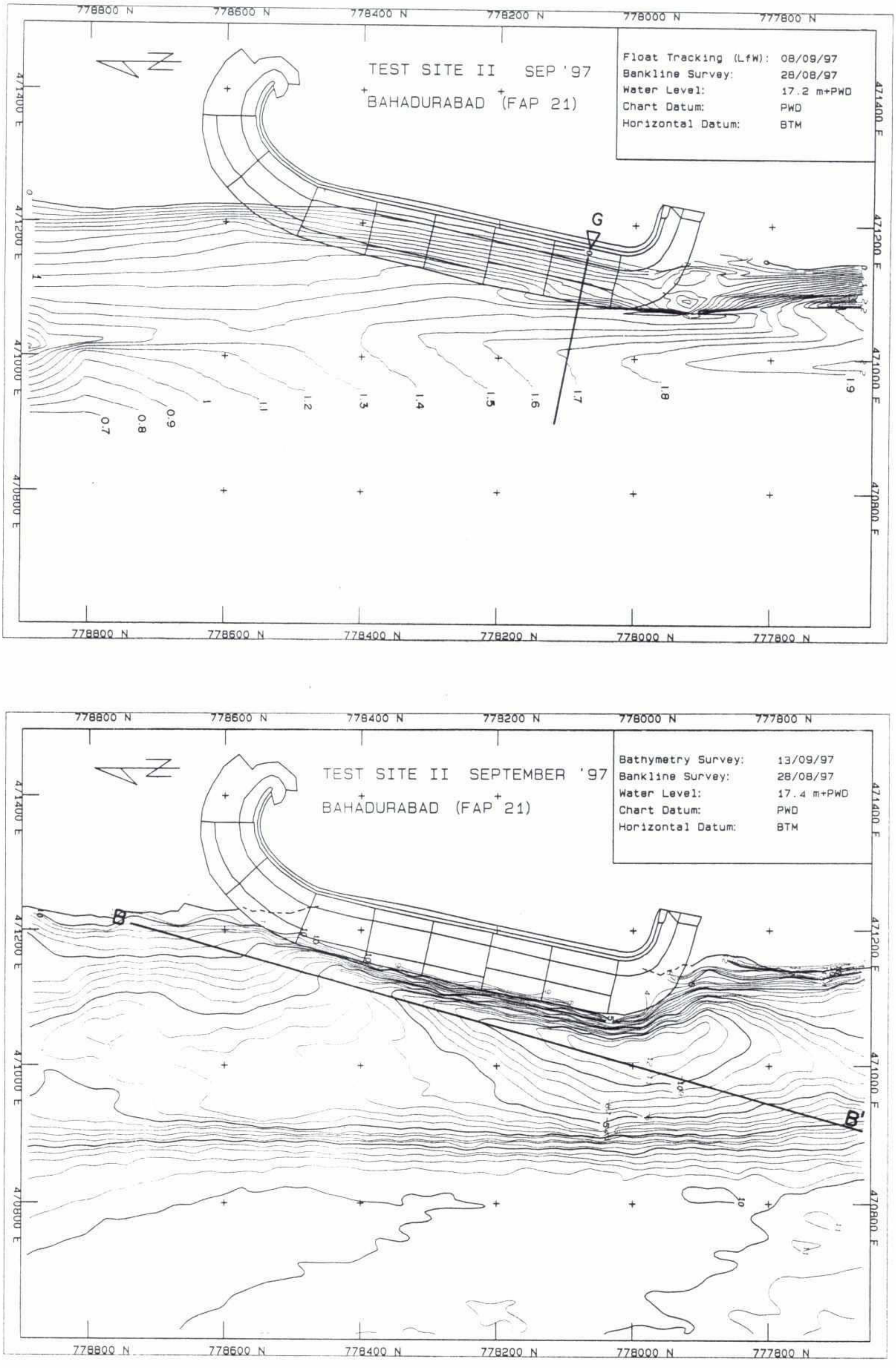
FIG. 4.8

88



BAHADURABAD TEST SITE	**FAP – 21**
Cross-section G, August 1997	**BANK PROTECTION PILOT PROJECT**
	FIG. 4.9

84



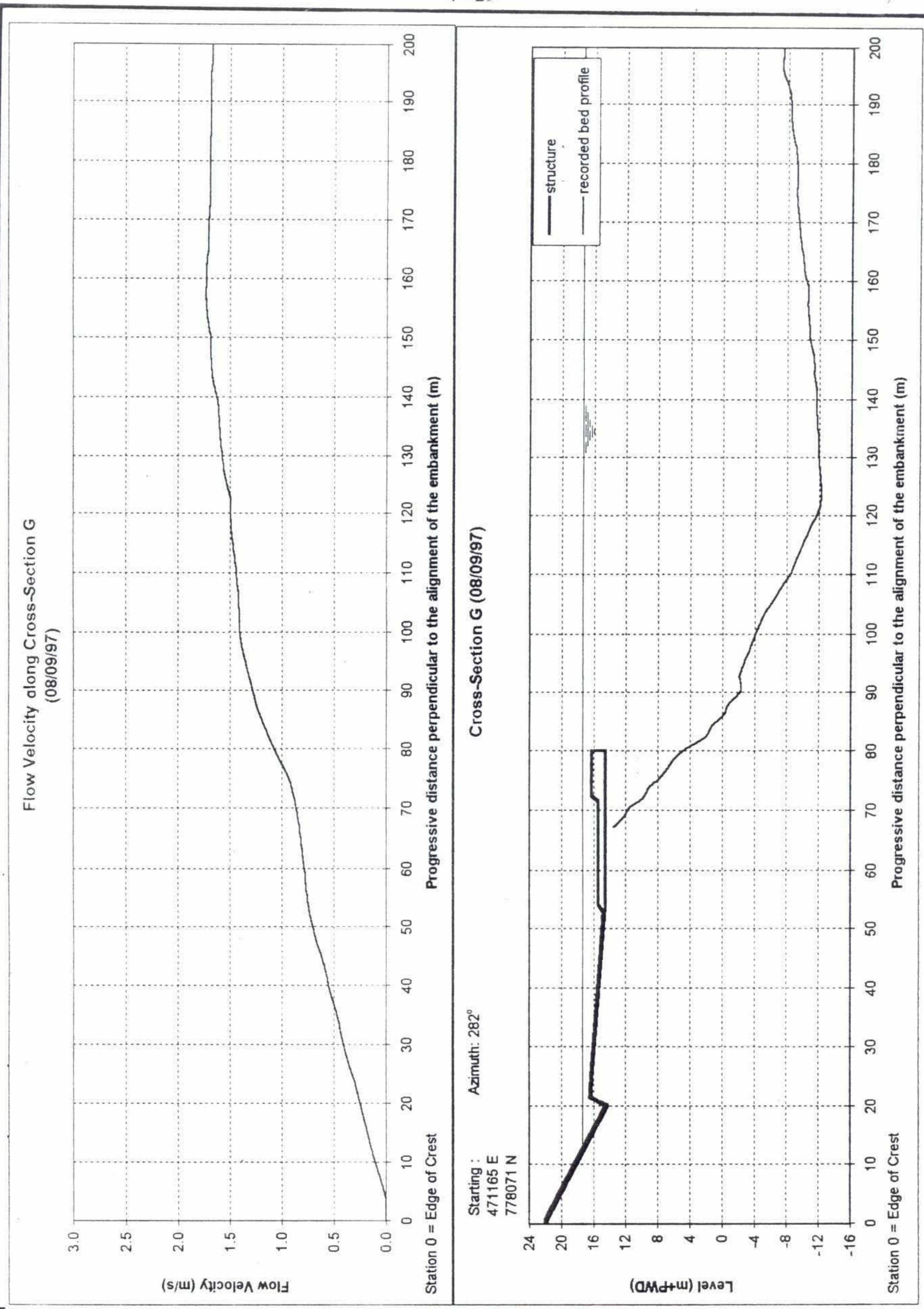
BAHADURABAD TEST SITE
Flow velocity model and bathymetry
September 1997

FAP - 21

BANK PROTECTION PILOT PROJECT

FIG. 4.10

89



BAHADURABAD TEST SITE

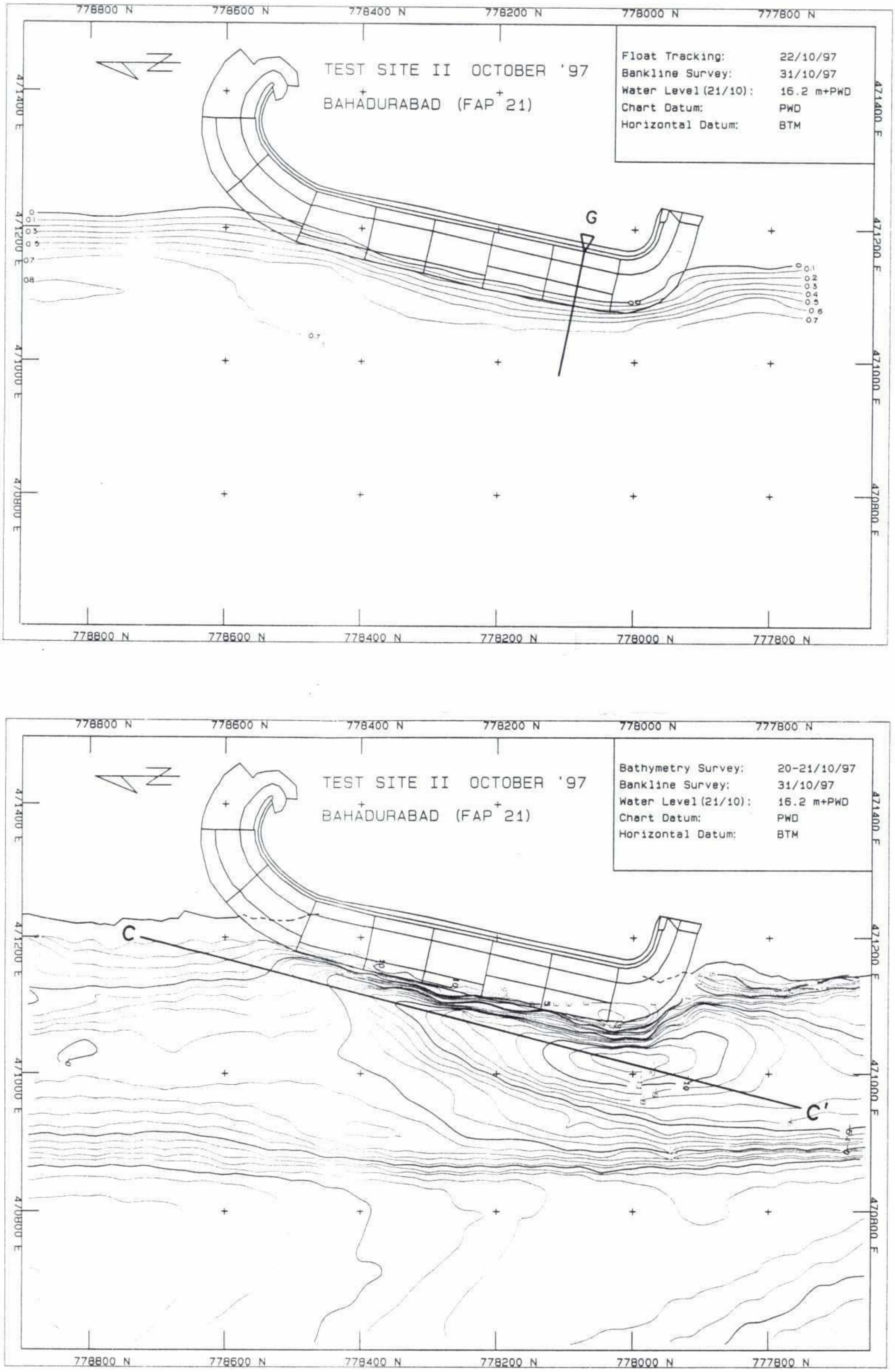
Cross-section G, September, 1997

FAP – 21

BANK PROTECTION PILOT PROJECT

FIG. 4.11

8b

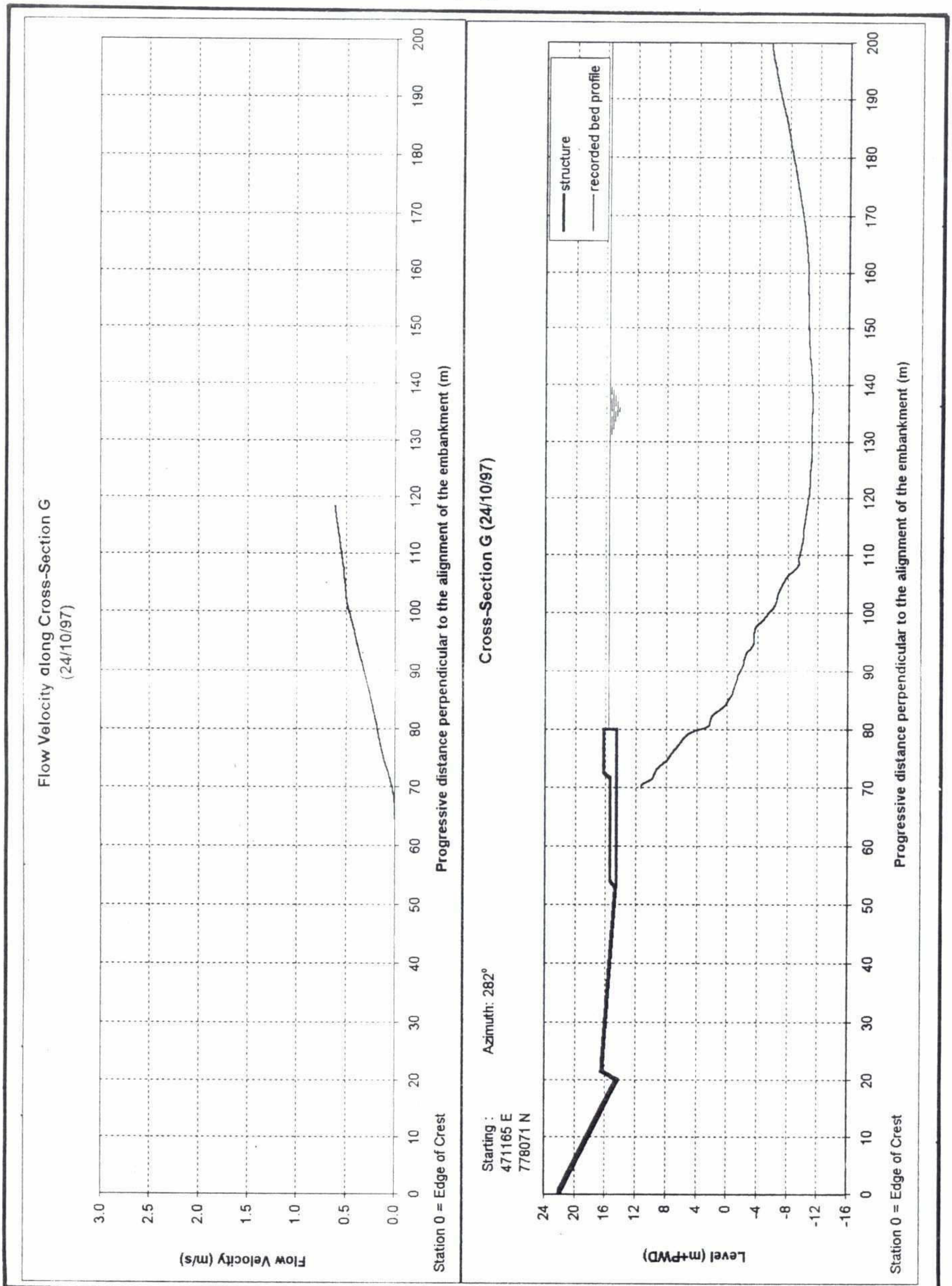


BAHADURABAD TEST SITE
Flow velocity model and bathymetry
October 1997

FAP - 21
BANK PROTECTION PILOT PROJECT

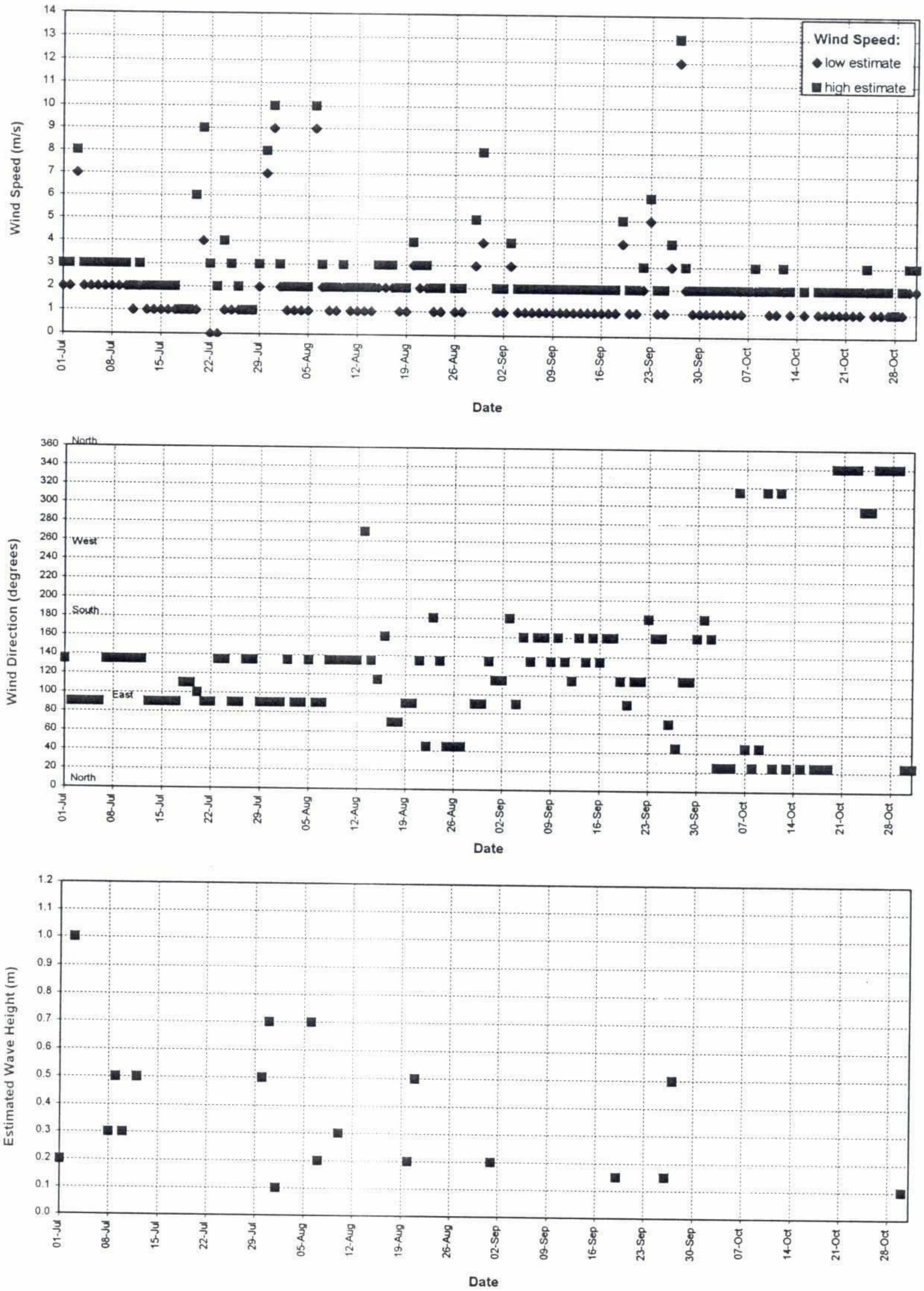
FIG. 4.12

87



BAHADURABAD TEST SITE Cross-section G, October 1997	**FAP – 21** **BANK PROTECTION PILOT PROJECT** **FIG. 4.13**

80



BAHADURABAD TEST SITE
Wind speed, wind Direction and estimated wave height

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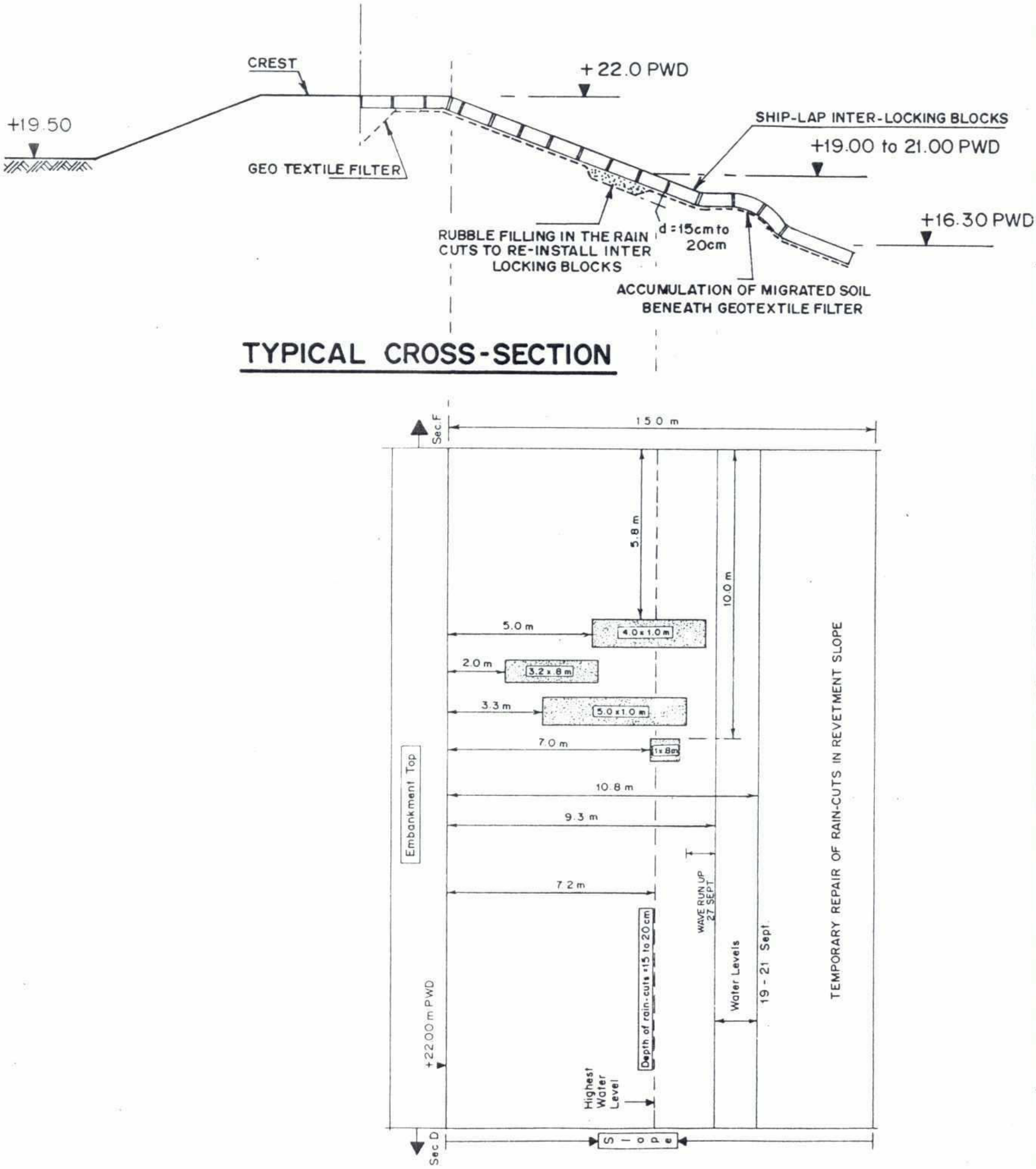
FIG. 4.14

a2



BAHADURABAD TEST SITE
Rain cuts under top layer of concrete slabs, Section E

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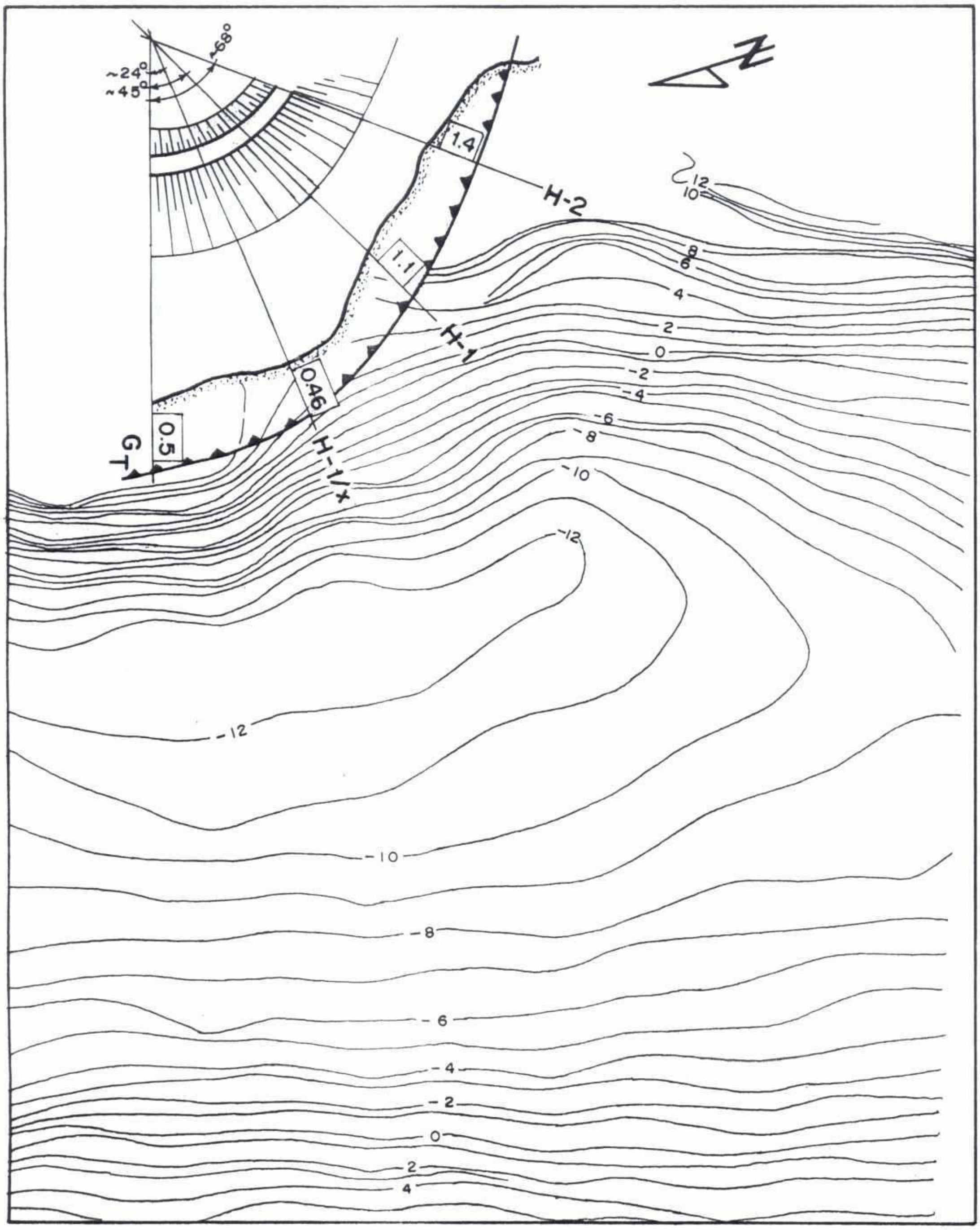
FIG. 4.15

42



LEGEND

CONTOUR LINE (m PWD)

BANKLINE 18.02.98

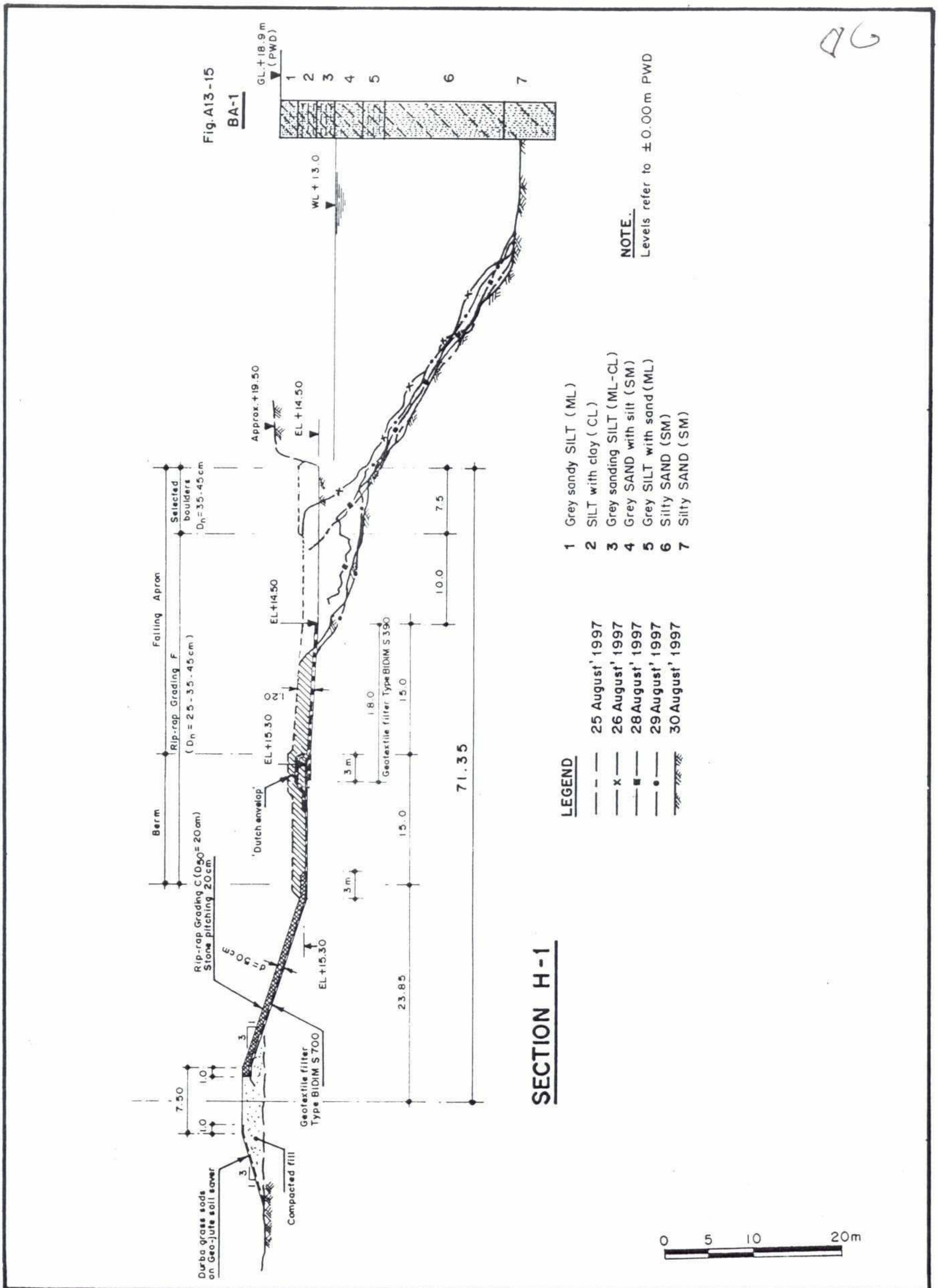
SLOPE OF BANK
FROM 7 to 14.5 m PWD

ORIGINAL TOE
FALLING APRON

SCALE

0 10 20 40 60 80 100m

BAHADURABAD TEST SITE Steep slopes falling apron Section H	**FAP - 21** BANK PROTECTION PILOT PROJECT **FIG. 4.16**



BAHADURABAD TEST SITE

Cross-section H-1 with falling apron

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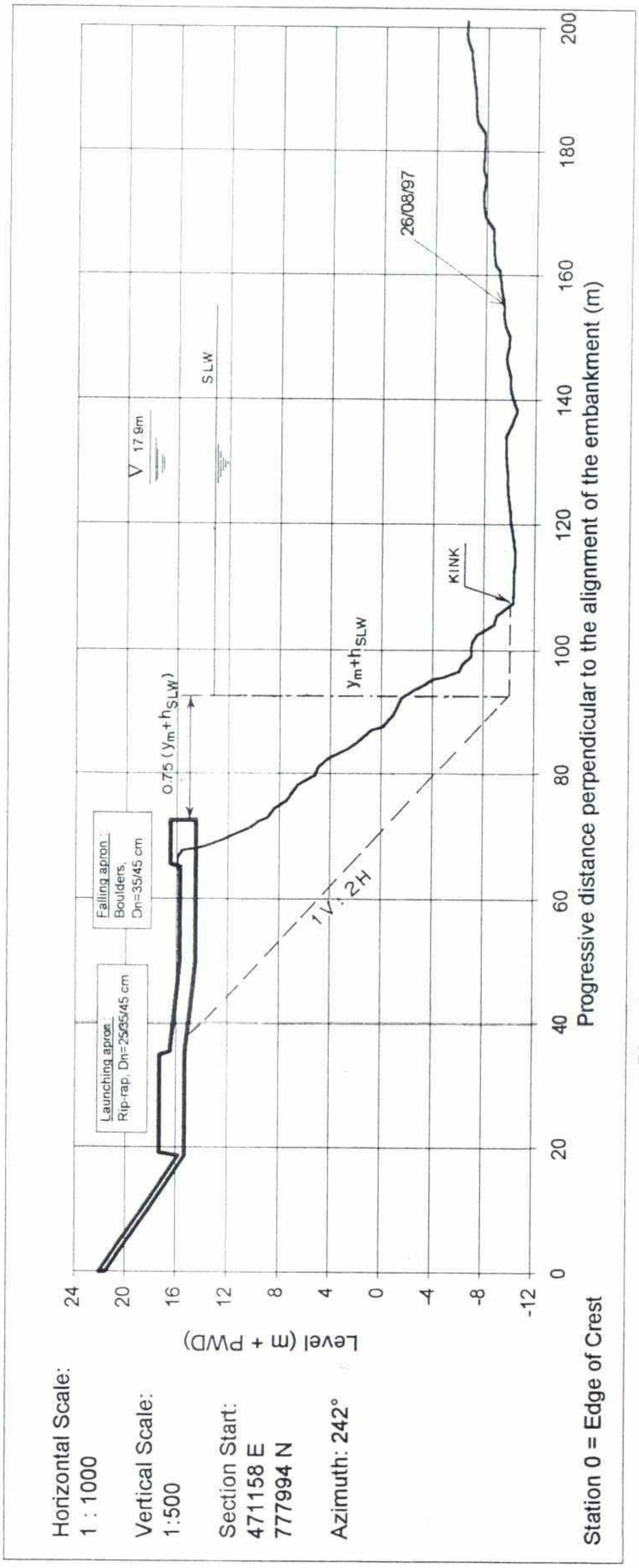
FIG. 4.17



BAHADURABAD TEST SITE

Cross-section H-1, on August 26, 1997

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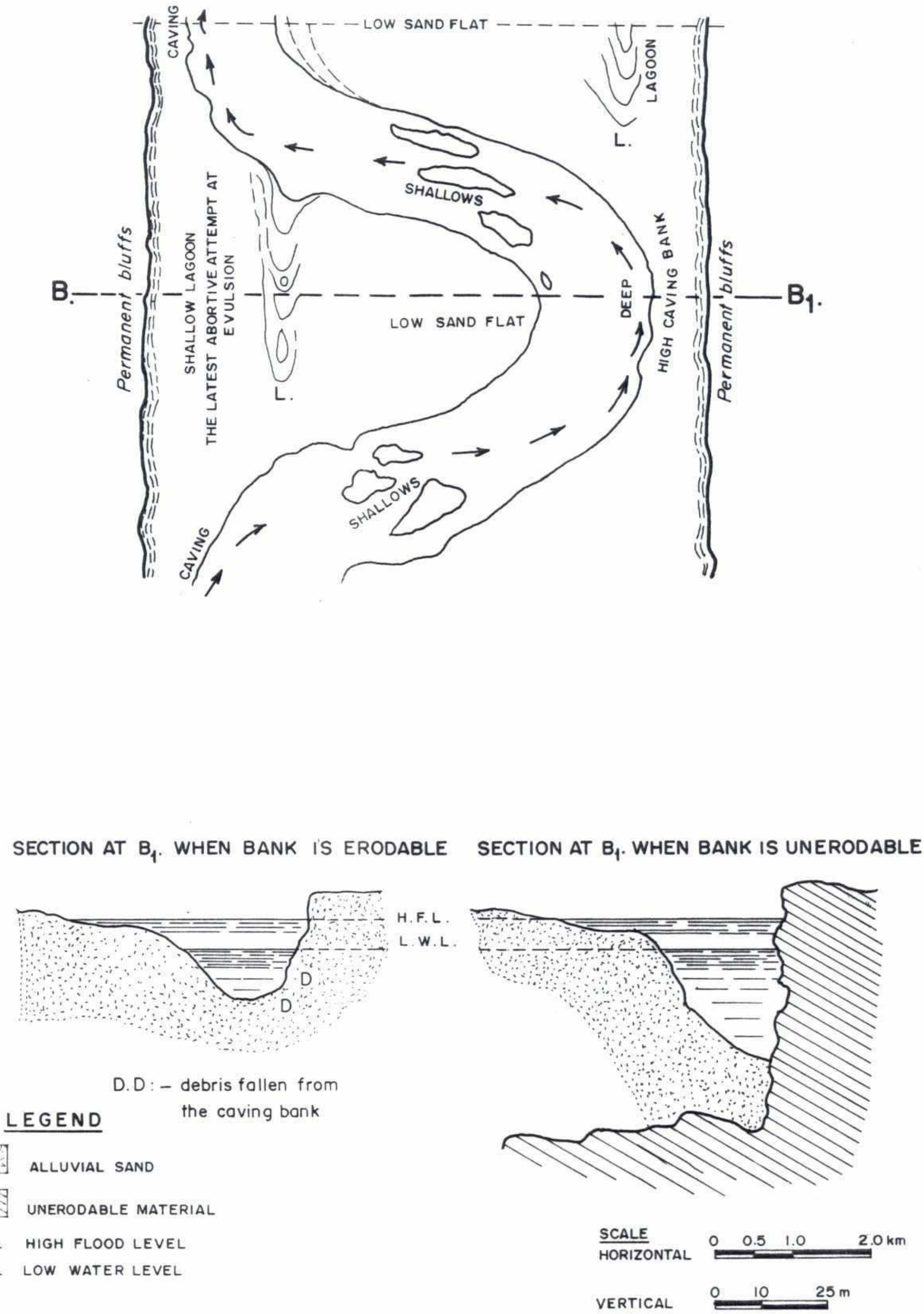
FIG. 4.18



BAHADURABAD TEST SITE
Sketch plan and sections showing action at a river bend

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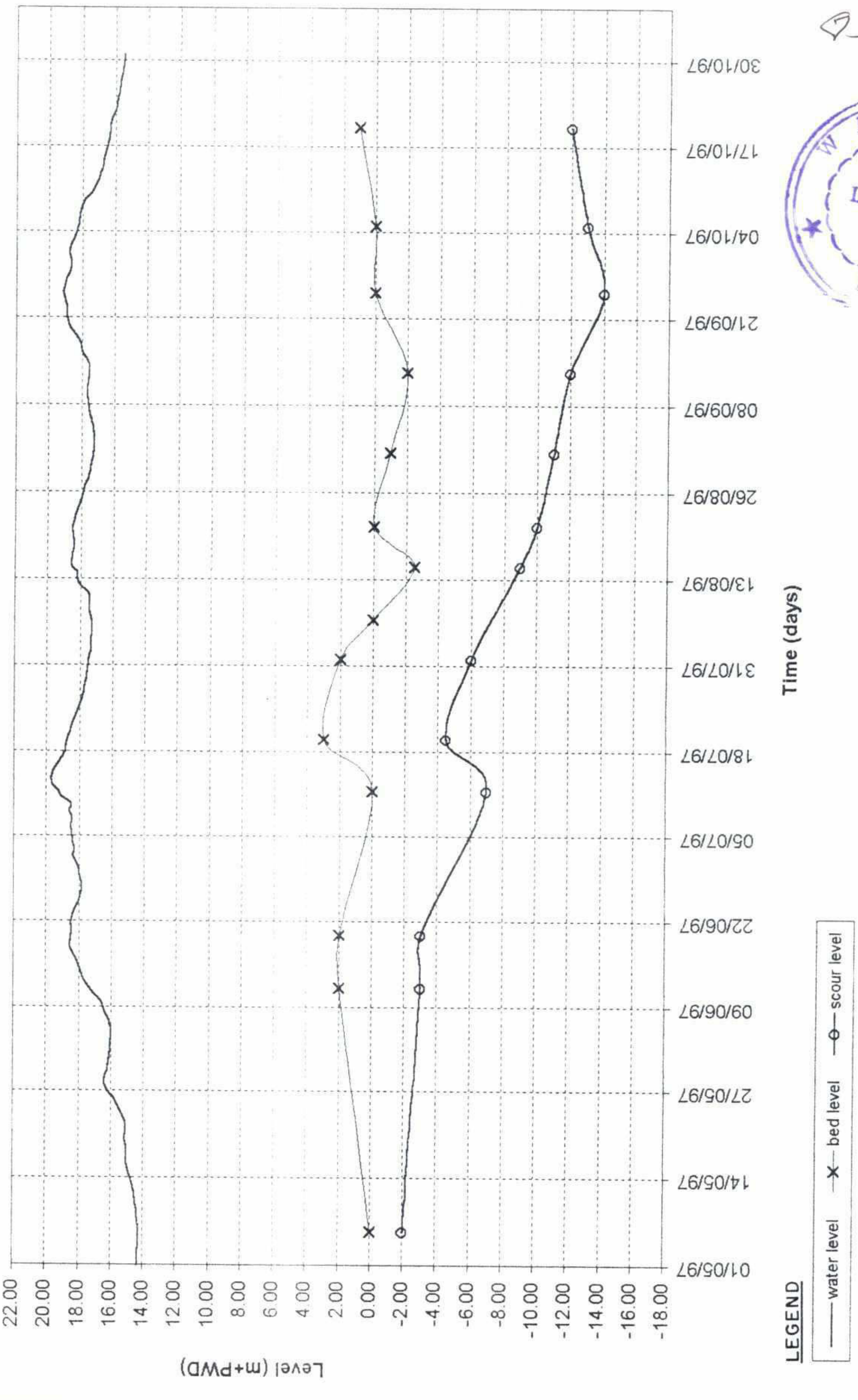
FIG. 4.19



BAHADURABAD TEST SITE
The bed level of approach channel and Maximum scour depth as function of time

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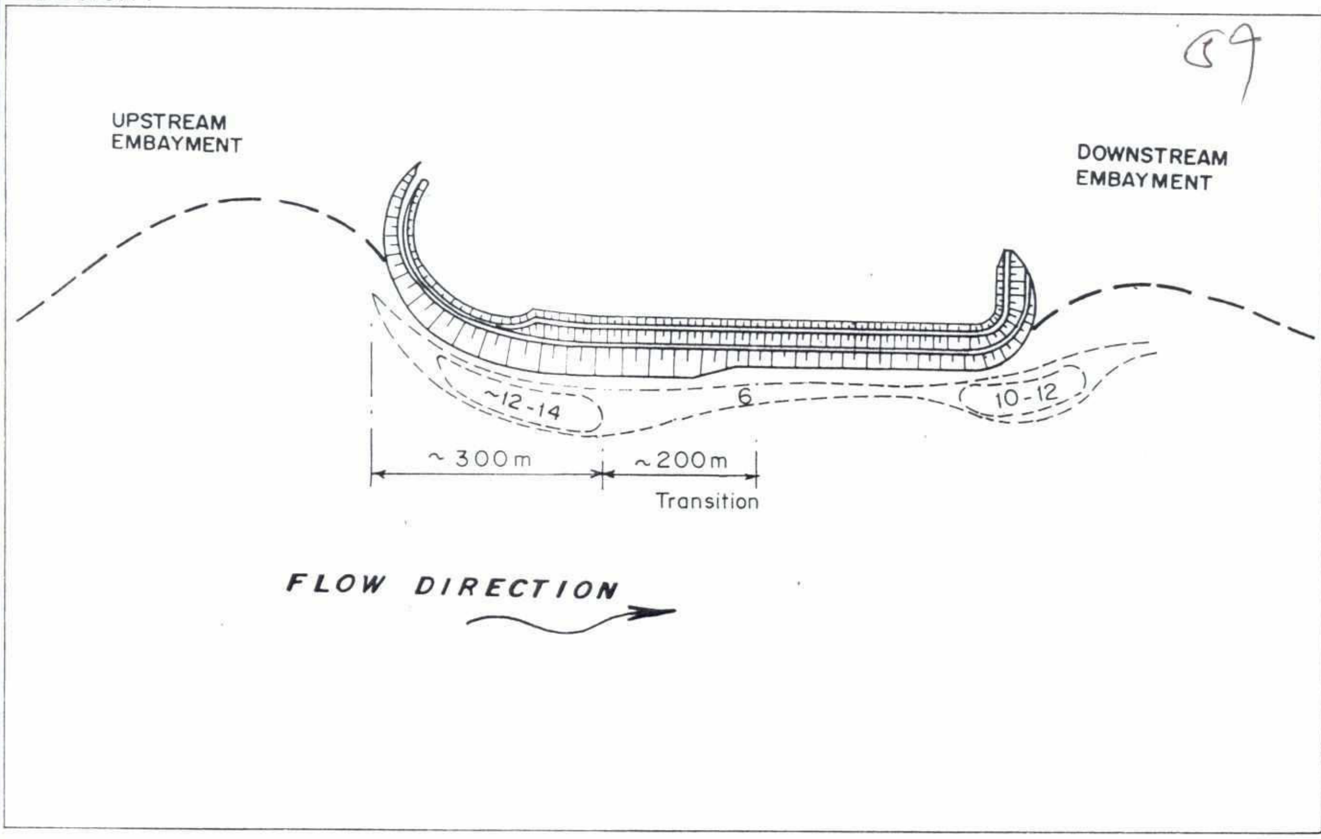
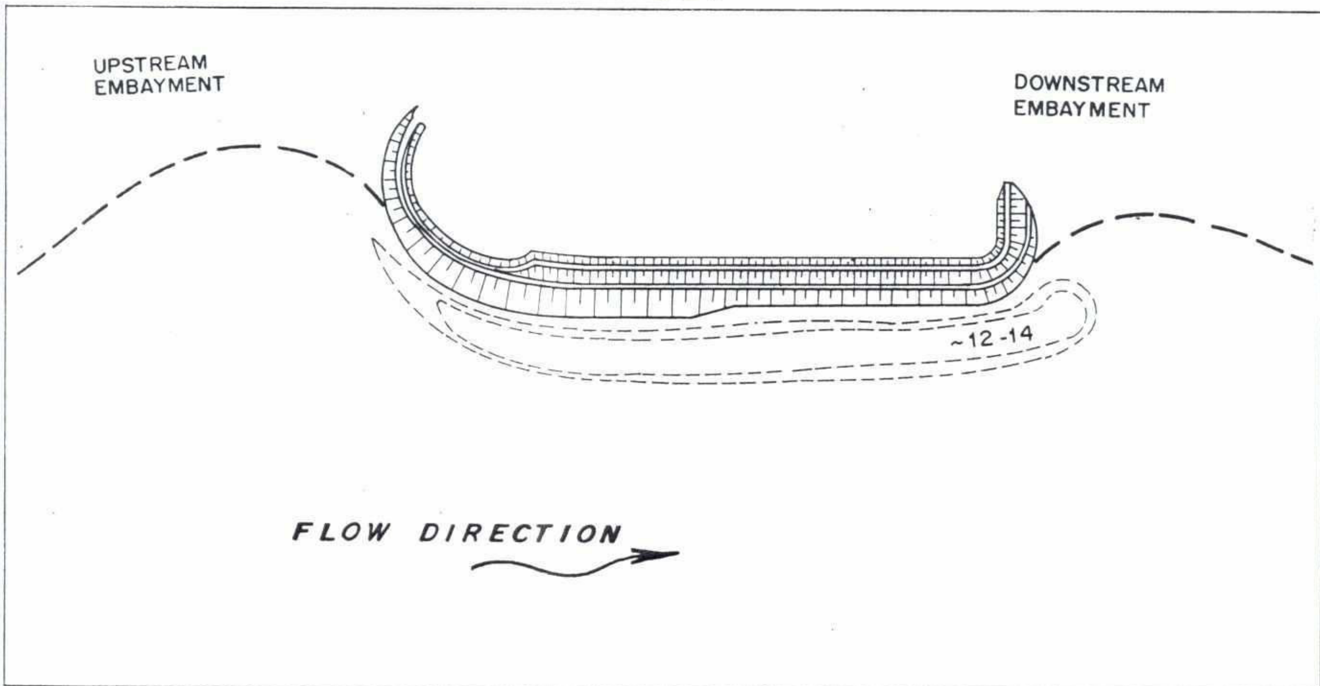
FIG. 4.20

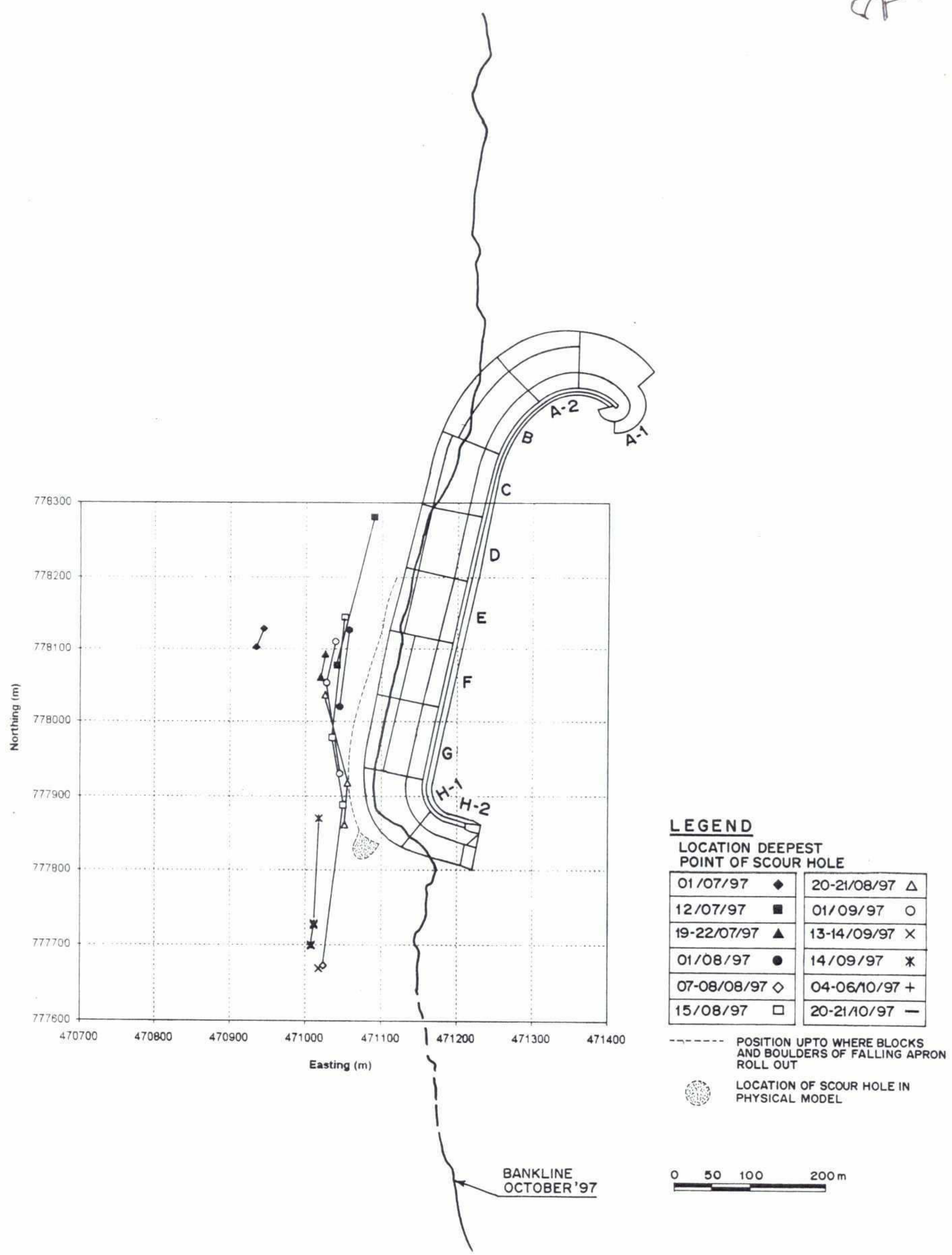
DESIGN :



MODIFICATION AFTER 1 YEAR MONITORING :



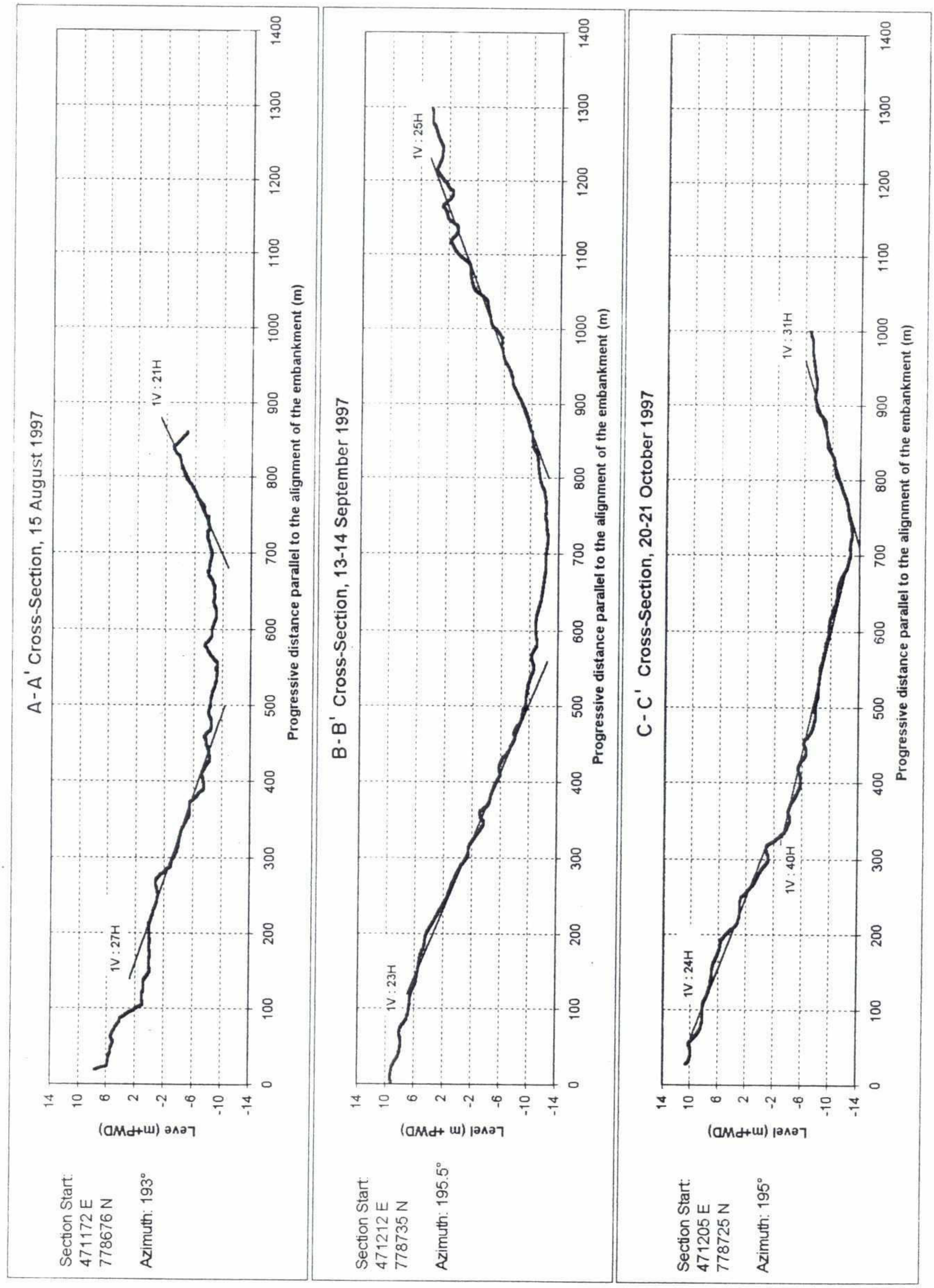
BAHADURABAD TEST SITE	**FAP – 21**
Design scour pattern and bankline	**BANK PROTECTION PILOT PROJECT**
	FIG. 4.21



BAHADURABAD TEST SITE
Location deepest point in scour hole at Bahadurabad

FAP - 21
BANK PROTECTION PILOT PROJECT

FIG. 4.22



BAHADURABAD TEST SITE

Slopes in longitudinal section of scour hole

FAP - 21

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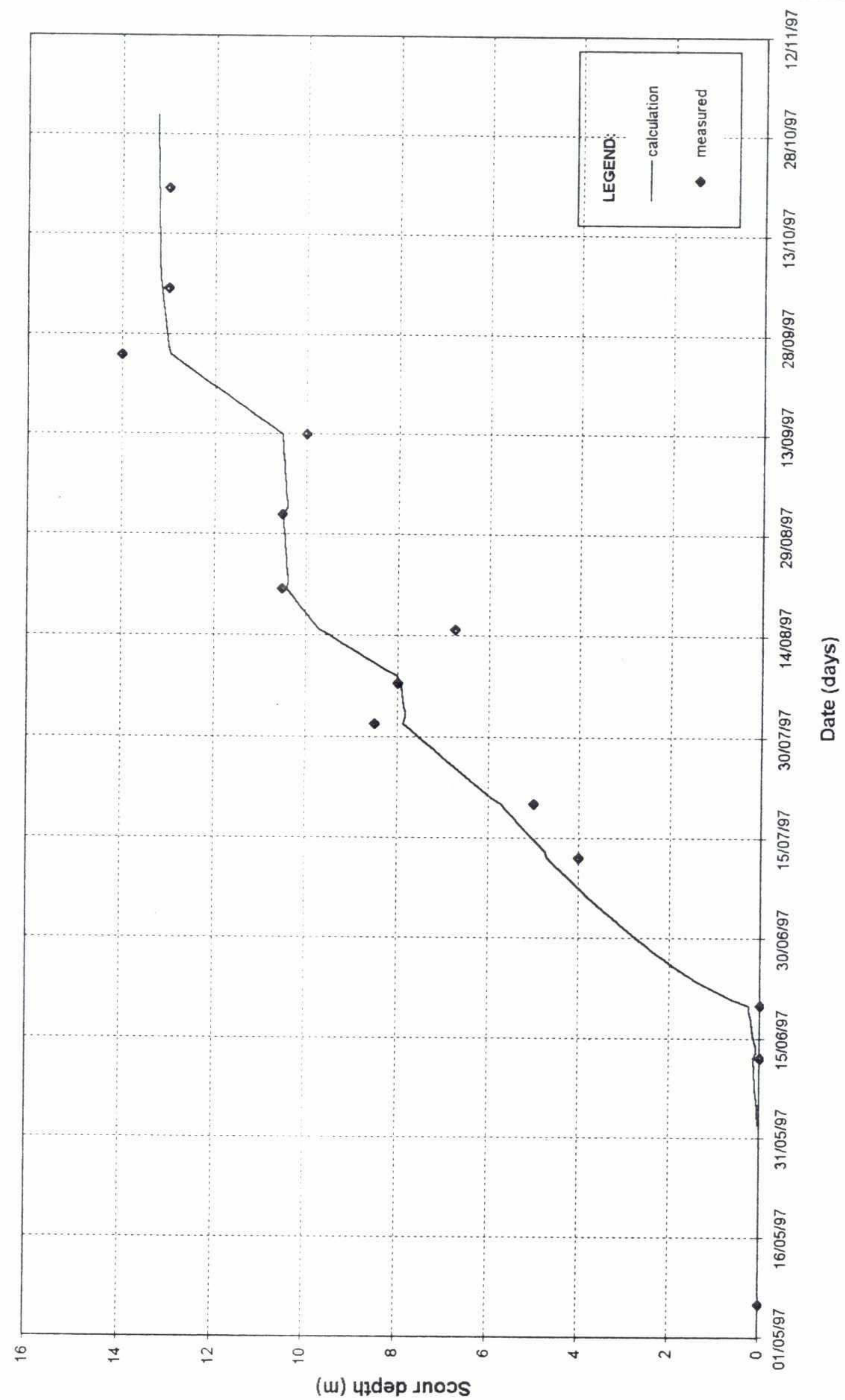
FIG. 4.23

40



BAHADURABAD TEST SITE

Calculation maximum scour depth

FAP – 21

BANK PROTECTION PILOT PROJECT

FIG. 4.24

5 STRUCTURAL OBSERVATIONS - DAMAGES

During the monsoon season 1997 the falling aprons and partly the launching aprons started to function as expected. No significant damages were observed on the sloped embankment apart from the rain-cuts in Section E.

In February 1998 a detailed technical assessment of all sections has been carried out. The results of visual observations and physical inspections at some selected locations are presented in Annex F supported by a photodocumentary and some laboratory tests.

In March 1998 measurements have been performed with a side scan sonar and subbottom profiler in front of the test structure in order to determine the spatial distribution of the materials used for the construction of the falling and launching aprons. The results of this survey have been presented as "Sonar and Subbottom Profiler Survey Report" in November 1998. Relevant results / information are also mentioned in the section-assessment sheets (see Annex F.2). A map with all side scan sonar results is presented in Annex F.3.

6 ADAPTATION AND REPAIR WORKS

Based on the results of all observations and investigations during and after the flood 1997 repair works were only necessary in Section E-2 and adaptation work in Section H.

To stabilize the slope of the revetment above the berm in Section E-2 the cover layer and filter layer had to be removed. The rain-cuts were filled up and the composite geotextile filter, which had been placed with the wrong side up during the construction phase, was replaced now with the right dark / coarse side down. On that a 5 cm thick filter layer of khoa was placed and finally the cc-slabs.

In Section H a supplemental falling apron of cc-blocks with a width of about 7.5 m was placed on the remaining part of the original falling apron following the bankline as existing in March 1998. The bottom of the slope was filled up by boulders grade E/F and the existing slope protected by a fill of cc-blocks of 30 and 40 cm. For details see Annex G.

7 CONCLUSIONS

The analysis of the monitoring data from the Revetment Test Structure during the 1997 monsoon has resulted in the following conclusions:

(i) Hydraulic Load:

- The hydraulic load on the revetment was below the design load by flow velocities, waves and water levels;
- The highest flow velocities occurred just before a peak water level was reached, and
- The flow velocity distribution in front of the revetment structure and along a cross section perpendicular to the main flow direction confirmed the design flow distribution.

(ii) Revetment Slope:

- In Section E rain-cuts have been detected underneath the geotextile. In the course of the investigations it was found out that the geotextile was wrongly placed (upside down). The rain-cuts could also be caused by wave action and because the weight of the blocks (ship-lap type) was too low. Beam-action between the slabs had reduced the pressure by the slabs on the geotextile. According to the results of the filter test investigation this low pressure causes rain-cuts damage under the geotextile.

(iii) Launching Apron:

- During the monsoon 1997 only the launching aprons in Section B and C started to function. No irregularities could be found at Section B (dumped cc-blocks). The articulated Reno-mattress in Section C had adjusted itself very good during the process of erosion and were flexible to the sub-soil forming a good bank protection. Only some single cages around the waterline seemed to have burst due to too heavy stresses. But it had also to be considered that one or two of them had been opened by local people to get access to the stone filling.

(iv) Falling Apron:

- The erosion rate of the falling apron reduced from 0.2 m/day at the start to 0.1 m/day which is the erosion rate of the scour hole. When the scour hole started to silt up also the falling apron silted up. The slow erosion rate of the scour hole had reduced the risk of larger slides;
- Two small slides or slips had occurred in the cohesive upperlayer. It is recommended to remove in future this upperlayer before the construction of the apron, or to increase the thickness of the apron to compensate loss of material in a slide;
- The slope of the falling apron face is on average 1V:2H with a good coverage if the thickness of the apron is 6 to 7 cc-blocks. This average slope confirms the design slope of 1V:2H;
- Steep slopes occurred near the downstream termination 1V:1.6H, with an unprotected bluff where the thickness of the apron was only 2 cc-blocks or boulders, and
- Falling apron of geo-containers did not provide a good coverage of the vertical bluff near SLW, but since the eroded river bank remained stable since August 1997 it can be assumed that the scattered containers protect the river bed from further erosion.

(v) Scouring:

- The maximum scour depth near the downstream termination was about 2 to 4 m more than the design scour depth. Therefore, the design scour depths have been modified. The location of the scour hole near the downstream termination was close to the location in the physical model tests and the assumed location in the design of the revetment structure;

- In the physical model investigations the test programme did not include a test with an oblique flow attack by a design channel. Only a smaller channel had been tested;
- The method to determine the design channel as an envelope curve of all surveyed channels had very well predicted the size of the approach channel of the scour hole during the 1997 monsoon, and
- A calculation method for the maximum scour depth has been calibrated.

The flow velocities near the revetment structure are an important part of the hydraulic load to the structure. Therefore, float tracking near the revetment structure will be made with more floats to determine the flow velocity distribution in more detail. Also more measurements with the Valeport flow meter will be made, especially during high flow velocities in periods of rising water levels.

The descriptions of the daily visual observations by the monitoring team are valuable information on the behaviour of the revetment test structure. However, the accuracy of these observations should be improved and more uniform descriptions are required.

The wave attack on the revetment will also be measured during storms.

Clo6

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