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MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANIZATION

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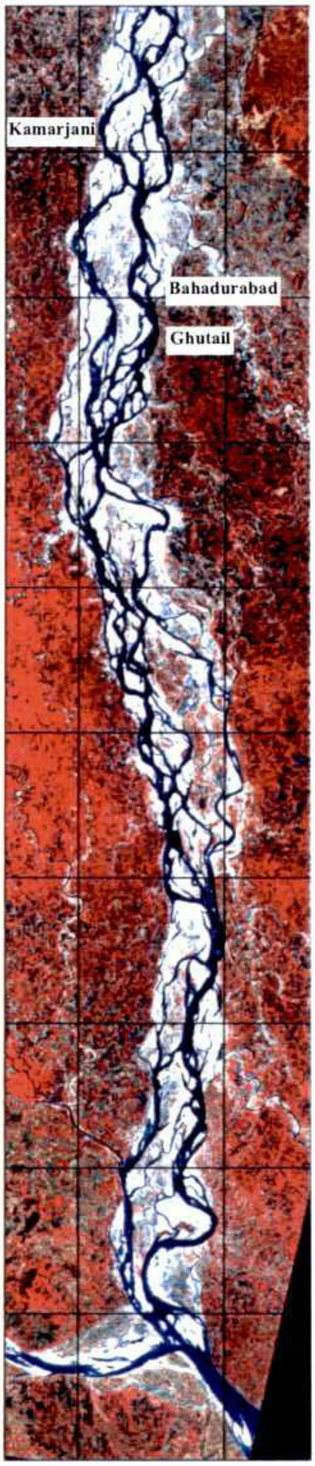

BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT FAP 21/22

FINAL PROJECT EVALUATION REPORT

VOLUME VI

Annex 10: The Revetment Test Structure; Monitoring Report

Annex 11: The Revetment Test Structure; Evaluation of Hydraulic Loads and River Response

DECEMBER 2001

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

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In association with:
BANGLADESH ENGINEERING & TECHNOLOGICAL SERVICES LTD. (BETS)
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BANK PROTECTION AND RIVER TRAINING (AFPM)
PILOT PROJECT FAP 21/22

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MAY 2001

CONTENTS OF FINAL PROJECT EVALUATION REPORT

Volume I	Main Report	
	Part A:	Bank Protection Pilot Project FAP 21
	Part B:	River Training (AFPM) Pilot Project FAP 22
Volume II	Annex 1	Morphological Investigations
	Annex 2	Socio-Economic Aspects
	Annex 3	Ecological Assessment
Volume III	Annex 4	The Groyne Test Structure; Design Report
	Annex 5	The Groyne Test Structure; Procurement and Construction Report
Volume IV	Annex 6	The Groyne Test Structure; Monitoring Report
	Annex 7	The Groyne Test Structure; Evaluation of Hydraulic Loads and River Response
Volume V	Annex 8	The Revetment Test Structure; Design Report
	Annex 9	The Revetment Test Structure; Procurement and Construction Report
Volume VI	**Annex 10**	**The Revetment Test Structure; Monitoring Report**
	Annex 11	**The Revetment Test Structure; Evaluation of Hydraulic Loads and River Response**

8

BANK PROTECTION PILOT PROJECT
FAP 21/22

Important General Remark

The results presented and discussed in the Annexes of the Final Evaluation Report of the Bank Protection Pilot Project provides the state of the studies during the course of analysis and writing by the individual project partners. After a final review subsequent to the completion of the Annexes during the concluding stages of the Main Reports and the Guidelines and Manual some modifications and adjustments were felt necessary, also for covering more generalized structural measures in addition to the given case studies. For that reason, with respect to design formulae, recommended structure types, etc., reference should be made to the Main Reports and the Guidelines and Design Manual only.

DECEMBER 2001

BANK PROTECTION PILOT PROJECT

FAP 21

FINAL PROJECT EVALUATION REPORT

ANNEX 10

THE REVETMENT TEST STRUCTURE; MONITORING REPORT

MAY 2001

FAP 21 - BANK PROTECTION PILOT PROJECT

FINAL PROJECT EVALUATION REPORT

ANNEX 10

Table of Contents

			Page
List of Acronyms			A-1
Glossary			G-1
SUMMARY			0-1
1	**INTRODUCTION**		**1-1**
	1.1	INTRODUCTION TO THE PROJECT	1-1
	1.2	SCOPE OF THE PRESENT ANNEX	1-1
	1.3	OVERVIEW OF THE TEST SITE AND LAYOUT OF THE STRUCTURE	1-1
	1.4	OBJECTIVES OF THE MONITORING	1-2
2	**ORGANISATION/IMPLEMENTATION**		**2-1**
	2.1	ORGANISATION OF THE MONITORING TEAM	2-1
	2.2	LOGISTICS	2-1
	2.3	GEODESY	2-1
	2.4	INSTRUMENTATION	2-1
	2.5	WATER LEVEL GAUGES	2-2
3	**FIELD SURVEYS**		**3-1**
	3.1	BATHYMETRY	3-1
	3.2	FLOW MEASUREMENTS	3-1
	3.3	LAND SURVEYS	3-2
	3.4	SIDE SCAN SONAR AND SUBBOTTOM SURVEYS	3-2
		3.4.1 Side Scan Sonar Description	3-2
		3.4.2 Subbottom Profiler Description	3-3
		3.4.3 System Configuration aboard Survey Boat	3-3
4	**THE REVETMENT TEST STRUCTURE**		**4-1**
	4.1	SUMMARY	4-1
	4.2	BATHYMETRY AND FLOW CONDITIONS	4-3
	4.3	CROSS SECTIONS	4-15
	4.4	DETAILED FLOW INVESTIGATIONS	4-35
		4.4.1 Advanced DGPS Float Tracking	4-35
		4.4.2 Current Point Measurements	4-35

9

				Page
5	**APRON INVESTIGATIONS**			**5-1**
	5.1	SIDE SCAN SONAR RESULTS		5-1
	5.2	SUBBOTTOM PROFILING		5-4
		5.2.1	Summary	5-4
		5.2.2	Results and Interpretations	5-4
	5.3	DIVING INSPECTIONS		5-22
REFERENCES				R-1

Page

LIST OF TABLES

Table 2.3-1:	Definition of water levels at Bahadurabad	2-1
Table 2.4-1:	Instrumentation specifications	2-2
Table 2.5-1:	Location and operation period of water level gauges	2-2
Table 5.2-1:	Location of objects detected by the subbottom profiler	5-4

LIST OF FIGURES

Fig. 1.3-1:	Jamuna overview at Test Site-II in January 1999	1-3
Fig. 1.3-2:	Layout of Revetment Test Structure	1-4
Fig. 3.4-1:	Side scan sonar towfish	3-2
Fig. 3.4-2:	System configuration aboard Obelix for SSS and SBP surveys	3-4
Fig. 4.1-1:	Bathymetry in June 1995	4-2
Fig. 4.1-2:	Bathymetry and flow in September 1996	4-2
Fig. 4.2-1:	Bathymetry and flow in May 1997	4-4
Fig. 4.2-2:	Bathymetry and flow in June 1997	4-4
Fig. 4.2-3:	Bathymetry and flow in June 1997	4-5
Fig. 4.2-4:	Bathymetry and flow in July 1997	4-5
Fig. 4.2-5:	Bathymetry in August 1997	4-6
Fig. 4.2-6:	Bathymetry and flow in August 1997	4-6
Fig. 4.2-7:	Bathymetry in September 1997	4-7
Fig. 4.2-8:	Bathymetry and flow in September 1997	4-7
Fig. 4.2-9:	Progressive bankline downstream of Test Site in 1997	4-8
Fig. 4.2-10:	Bathymetry and flow in April 1998	4-9
Fig. 4.2-11:	Bathymetry and flow in May 1998	4-9
Fig. 4.2-12:	Bathymetry and flow in June 1998	4-10
Fig. 4.2-13:	Bathymetry in July 1998	4-10
Fig. 4.2-14:	Bathymetry in August 1998	4-11
Fig. 4.2-15:	Bathymetry and flow in September 1998	4-11
Fig. 4.2-16:	Bathymetry and flow in October 1998	4-12
Fig. 4.2-17:	Bathymetry and flow in November 1998	4-12
Fig. 4.2-18:	Bathymetry and flow in March 1998	4-13
Fig. 4.2-19:	Bathymetry in May 1999	4-13
Fig. 4.2-20:	Bathymetry and flow in June 1999	4-14
Fig. 4.2-21:	Bathymetry and flow downstream of Test Site in June 1999	4-14
Fig. 4.2-22:	Bathymetry and flow in July 1999	4-15
Fig. 4.3-1:	Bathymetry cross section B in 1997	4-16
Fig. 4.3-2:	Bathymetry cross section C in 1997	4-17
Fig. 4.3-3:	Bathymetry cross section D in 1997	4-18
Fig. 4.3-4:	Bathymetry cross section E in 1997	4-19
Fig. 4.3-5:	Bathymetry cross section F in 1997	4-20
Fig. 4.3-6:	Bathymetry cross section G in 1997	4-21
Fig. 4.3-7:	Bathymetry cross section B in 1998, 1999	4-22
Fig. 4.3-8:	Bathymetry cross section C in 1998, 1999	4-23
Fig. 4.3-9:	Bathymetry cross section D in 1998, 1999	4-24

Page

Fig. 4.3-10:	Bathymetry cross section E1 in 1998, 1999	4-25
Fig. 4.3-11:	Bathymetry cross section E2 in 1998, 1999	4-26
Fig. 4.3-12:	Bathymetry cross section F in 1998, 1999	4-27
Fig. 4.3-13:	Bathymetry cross section G in 1998, 1999	4-28
Fig. 4.3-14:	Bathymetry cross section H1 in 1998, 1999	4-29
Fig. 4.3-15:	Bathymetry cross section H2 in 1998, 1999	4-30
Fig. 4.3-16:	Steepest slope surveyed in Section B and C	4-31
Fig. 4.3-17:	Steepest slope surveyed in Section D and E	4-32
Fig. 4.3-18:	Steepest slope surveyed in Section F and G	4-33
Fig. 4.3-19:	Steepest slope surveyed in Section H-1 and H-2	4-34
Fig. 4.4-1:	Flow pattern at Test Site II in June 1997	4-37
Fig. 4.4-2:	Flow pattern at Test Site II in June 1997	4-38
Fig. 4.4-3:	Flow pattern at Test Site II in June 1997	4-39
Fig. 4.4-4:	Flow pattern at Test Site II in August 1997	4-40
Fig. 4.4-5:	Flow pattern at Test Site II in June 1998	4-41
Fig. 4.4-6:	Current point measurement in June 1998	4-42
Fig. 4.4-7:	Current point measurement at Section E1	4-43
Fig. 4.4-8:	Current point measurement at Section F	4-44
Fig. 5.1-1:	Side Scan Sonar Coverage March 1998	5-2
Fig. 5.1-2:	Side Scan Sonar Coverage March 1999	5-3
Fig. 5.2-1:	Subbottom cross section at Section B	5-6
Fig. 5.2-2:	Subbottom cross section at Section C	5-7
Fig. 5.2-3:	Subbottom cross section at Section D	5-8
Fig. 5.2-4:	Subbottom cross section at Section E1	5-9
Fig. 5.2-5:	Subbottom cross section at Section E2	5-10
Fig. 5.2-6:	Subbottom cross section at Section F	5-11
Fig. 5.2-7:	Subbottom cross section at section G	5-12
Fig. 5.2-8:	Subbottom cross section at section H1	5-13
Fig. 5.2-9:	Subbottom cross section at section H2	5-14
Fig. 5.2-10:	Subbottom cross section 10 m d/s of Section G	5-15
Fig. 5.2-11:	Subbottom cross section 20 m d/s of Section E2	5-16
Fig. 5.2-12:	Subbottom cross section 30 m u/s of Section F	5-17
Fig. 5.2-13:	Subbottom cross section 10 m u/s of Section F	5-18
Fig. 5.2-14:	Subbottom cross section 30 m d/s of Section F	5-19
Fig. 5.2-15:	Subbottom cross section 50 m u/s of Section G	5-20
Fig. 5.2-16:	Subbottom cross section 30 m u/s of Section G	5-21
Fig. 5.3-1:	Diving inspection at Section B	5-24
Fig. 5.3-2:	Diving inspection at Section C	5-25
Fig. 5.3-3:	Diving inspection at Sections D and E1	5-26
Fig. 5.3-4:	Diving inspection at Sections E2 and F	5-27
Fig. 5.3-5:	Diving inspection at Sections G and H1	5-28
Fig. 5.3-6:	Diving inspection at Section H2	5-29

ATTACHMENTS

Attachment-1: Logbook Form Sheet
Attachment-2: Hydrographs from June to September in 1995 to 1999
Attachment-3: Precipitation at Bahadurabad
Attachment-4: Data Archive System FAP 21
Attachment-5: Inventory of Monitoring Data

LIST OF ACRONYMS

BWDB	-	Bangladesh Water Development Board
DGPS	-	Differential Global Positioning System
DHW	-	Design High Water Level
DXF	-	AutoCAD compatible format of digital data
EGIS	-	Environmental and GIS Support Project for water sector planning (formerly FAP 19)
FAP	-	Flood Action Plan
GIS	-	Geographic Information System
GPS	-	Global Positioning System
HF	-	High Frequency
HP-GL/2	-	Hewlett Packard's standard graphics language for its plotters
MDL	-	Measurement Devises Engineering Limited
MSL	-	Mean Sea Level
PWD	-	Public Works Department (datum level)
SHW	-	Standard High Water
SLW	-	Standard Low Water
TPA	-	Test Pile Acceleration
TPI	-	Test Pile Inclination
UHF	-	Ultra High Frequency
VHF	-	Very High Frequency
WGS'84	-	World Geodetic System defined in 1984

22

GLOSSARY

TERM	DEFINITION
AutoCAD	Auto Computer Aided Design, Release 14 used in this Project
BTM	Bangladesh Transverse Mercator (common projection used in Bangladesh)
DTM	Digital Terrain Model
Gauge	A gauge in its simplest form consists of a vertical staff, graduated in centimeters and placed in such a position that the surface level of the water may be read from the scale at any time. The gauge must be connected to land elevations
Geodesy	Geodesy is the science, which deals with the investigations of the shape and dimensions of the earth's surface.
Geoid	Definition of earth surface assuming the continuation of the mean sea water level, depending on local gravity field.
GIS	Geographical Information System
Projection	Mathematics method to project spheroid coordinates (geographic coordinates) on a plain surface (grid coordinates)
PWD	Public Works Department reference for vertical co-ordinates
Soundings	Mathematically defined shape of earth, which fits best for a defined area. The spheroid is defined by two parameters (major axis and inverse flattering). The Everest Modified 1830 Spheroid is applied by this Project.
Transducer	Part of an Echo Sounder, which transmit and receive an acoustic pulse
REC-module	Memory Card for storage of digital data. The data can be transferred to a computer by a REC-module reader

SUMMARY

The monitoring of the Brahmaputra-Jamuna river provides information for all river engineering fields. This information is substantial for the planning, the design, the implementation and adaptation works of bank protection structures.

The structure at Bahadurabad was continuously monitored under FAP 21/22 from May '97 to the end of the year 1999. The collected monitoring data of 3 years were the basis to deepen the knowledge on the river engineering fields of the project. The data cover information about hydrography, hydraulic action, hydrology, topography and meteorology. During the monitoring period the survey and processing techniques were continuously developed to optimise the output. The techniques used in this project are described in Annex 6. Special investigations on the aprons in front of the revetment structure were carried out by side scan sonar and sub bottom measurements. These techniques are described in this Annex.

Bathymetry and float tracking surveys were the most appropriate methods to describe the prevailing river conditions. Using DGPS technology, a survey echo sounder and hydrographic PC-based software these surveys were carried out. Other types of surveys and data collection related to river engineering tasks practised in the project are presented here as well. Besides the collection of data the processing and mapping was an important topic of the monitoring. The graphical layout of the presentations should attract the user. Standard layouts were developed for better comparison of consequent surveys as they are presented in this Annex. It is also essential to define a well-organised database, which enables the user, an easy and quick access to the required data. The structure of the database and an inventory of all monitoring data are presented in the Appendices of this Annex.

To organise and to combine all river survey data from other projects and agencies in the past and in the future in one complex database remains a future topic for research. A geographical information system (GIS) related to river geo-data could be a suitable database for future research and analysis of rivers in Bangladesh.

1 INTRODUCTION

1.1 INTRODUCTION TO THE PROJECT

The objectives of the Project were 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 had been monitored from the monsoon season 1997 to the end of 2000. The final objective was 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 river 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 after their completion as well as preventive maintenance and adaptation of the structures is a must 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.

1.2 SCOPE OF THE PRESENT ANNEX

The present Annex 10 presents the findings of the monitoring of the Revetment Test Structure at Bahadurabad. The basis of this Annex are the yearly Monitoring and Adaptation Reports and the monthly Monitoring Reports (see list of references). It shall be noted that some differences to former reports of the Project are possible. Because of new experiences collected during the monitoring period and new developed processing software was necessary for some evaluations to refer to the raw data, which has been collected in the field. In case the existing results have been found doubtful, the data were checked, partly reprocessed and the results corrected accordingly, if necessary.

In general, the monitoring at Bahadurabad is comparable with the monitoring at Kamarjani. The same survey equipment and techniques have been used. A detailed description is given in Chapter 2 and 3 of Annex 6. The organisation/implementation of the monitoring at Bahadurabad Test Site and the field surveys, which differ from the monitoring at Kamarjani, are described in Chapter 2 and 3 of the present annex. As to recommendations for future monitoring of riverbank protection works reference is made to Chapter 8 of Annex 6.

The main scope of this Annex is to present the results of the monitoring at Bahadurabad for the period 1997 to 1999. The result presentation is separated in the bathymetry and flow conditions in front of the test structure (Chapter 4) and the special sub-water investigations on the falling aprons of the revetment structure (Chapter 5). A complete inventory of the collected monitoring data is provided in the Attachment 3, whereas Attachment 1 shows a logbook form sheet and Attachment 2 describes the data archive system used by FAP 21.

1.3 OVERVIEW OF THE TEST SITE AND THE LAYOUT OF THE STRUCTURE

The Revetment Test Structure is located at Kulkandi village just downstream from Bahadurabad ghat at the left bank of the Jamuna. Fig. 1.3-1 shows an overview of that area based on a Landsat image of

January 1999. A main channel bathymetry survey is super imposed illustrating the investigated area monitored within the FAP 21 Project.

The final layout of the structure is presented in Fig. 1.3-2. The total length of the structure is 662.5 m split up into 10 sections, each consisting of a sloped embankment 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 embankment revetment as well as for launching and falling aprons (Fig. 1.3-2).

1.4 OBJECTIVES OF THE MONITORING

The objectives are comparable with the monitoring programme at Test Site I at Kamarjani (see Annex 6). In addition, special sub-water investigations were carried out above the falling aprons. Side scan sonar in combination with seismic subbottom measurements were done in March 1998. In February/March 1999 the aprons were investigated again by side scan sonar measurements and in addition by diving operations. The intention of these surveys was to measure the actual extent of the falling apron and to check the structural condition of the aprons.

Beside these special investigations the functioning of the aprons was controlled by frequent depth cross section surveys. In cases, the structure was under flow attack during the monsoon period the cross sections surveys were done daily. They were defined at the centre of each section.

A monitoring station equipped with a unit of self-recording wind, wave and water level measurements was not available at Bahadurabad, therefore, visual observations were reported in the logbook (Attachment 1).

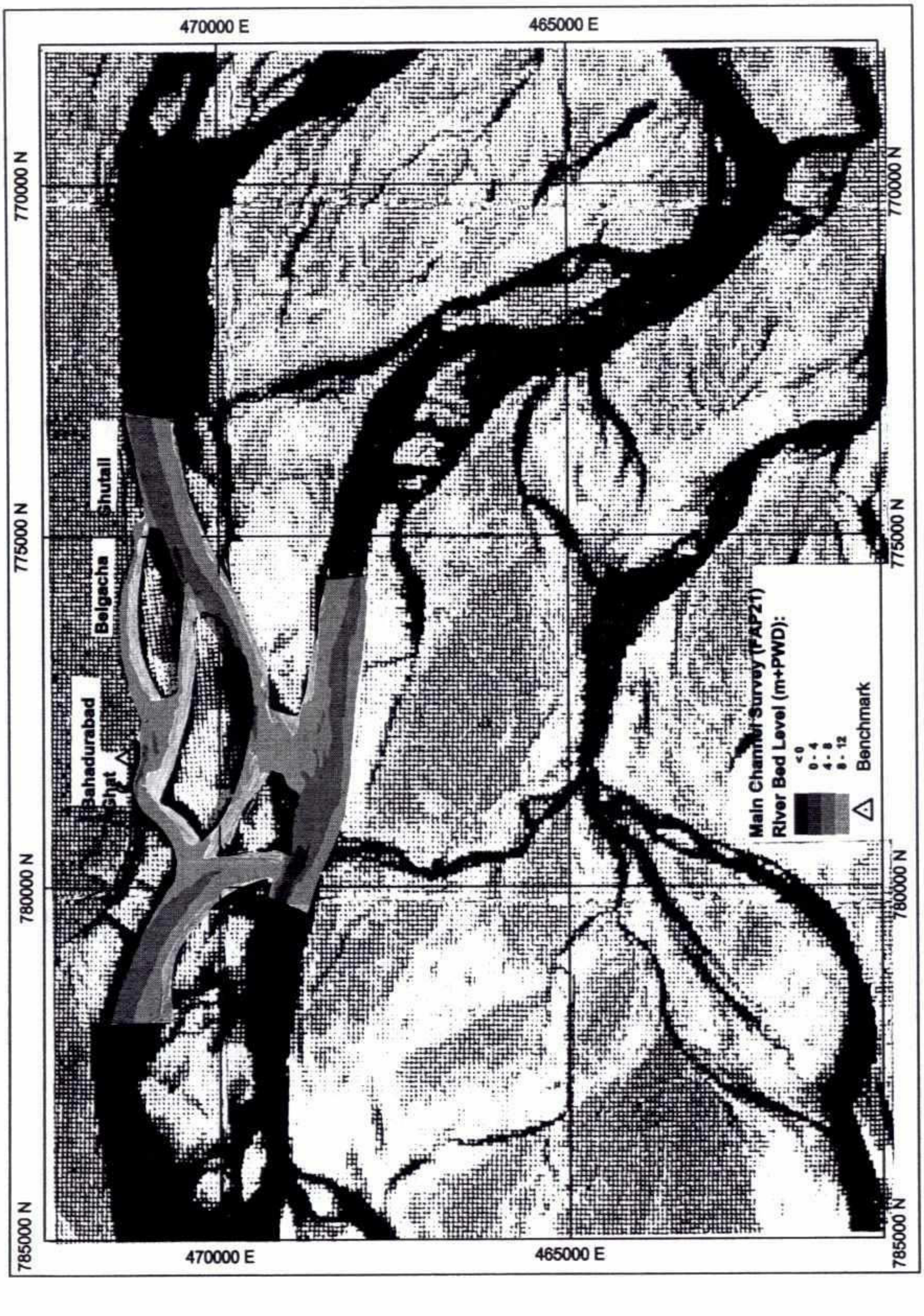


Fig. 1.3-1: Jamuna overview at Test Site II in January 1999

29

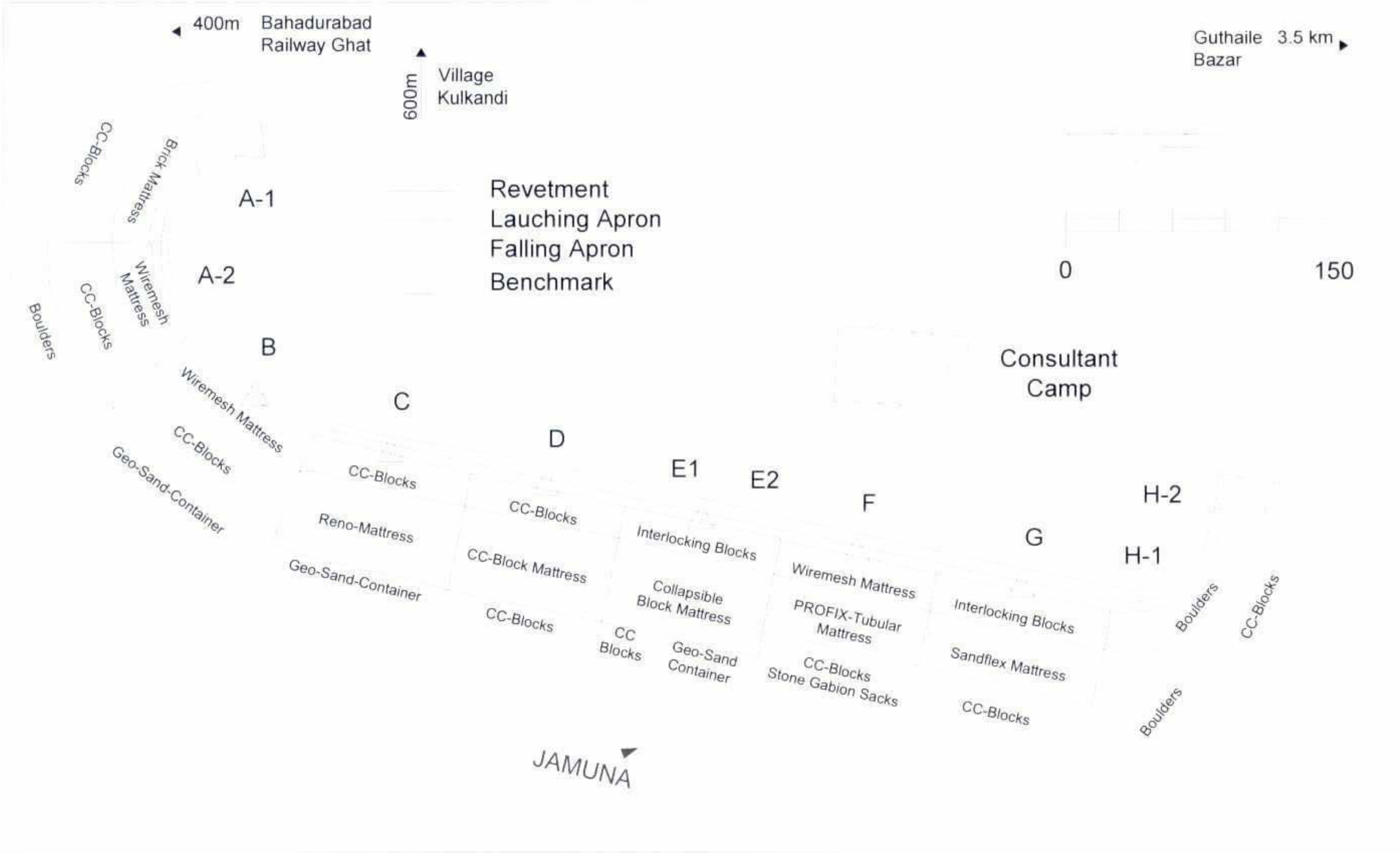


Fig. 1.3-2: Layout of Revetment Test Structure

21

2 ORGANIZATION/IMPLEMENTATION

2.1 ORGANIZATION OF THE MONITORING TEAM

The monitoring team is described in Annex 6. It was responsible for monitoring the two FAP 21 test sites at Kamarjani and Bahadurabad as well as for the monitoring of the FAP 22 activities.

2.2 LOGISTICS

The consultant base camp is located just 100 m behind the revetment structure (see Fig. 1.3-2). It consists mainly of bungalows to provide accommodation, office and storage facilities. After the closing of the base camp at Test Site I in 1999, four 20 feet containers were placed additionally at the Bahadurabad camp.

The communication between the base camp and the field survey teams was provided by VHF radio, to Dhaka head office by HF radio respectively. The base camp at Kamarjani was in range of the VHF radio as well.

The monitoring data could be pre-processed at site. Especially the cross-section surveys were processed on the same day as surveyed and the results were sent by courier and delivered at Dhaka head office the next morning.

2.3 GEODESY

As to the horizontal and vertical reference system used in this Project, reference is made to Annex 6.

The BTM co-ordinates were transferred from Kamarjani to Bahadurabad by DGPS within an accuracy of 3 m. From this origin a global net of benchmarks has been established and maintained during the complete monitoring period by the electronic total station. The relative accuracy of the benchmarks is estimated like at Kamarjani as $S_{x,y} = S_z = 2$ cm. The absolute net accuracy relative to the BTM grid is estimated around 5 m. A description of all existing benchmarks at the end of 1999 is given in Attachment 3 (Inventory of Monitoring Data).

The defined water levels for SLW, SHW and DHW at Bahadurabad are listed below.

Location	SLW	SHW	DHW
Test Site II Bahadurabad	13.3 m	18.8 m	21.1 m

Table 2.3-1: Definition of water levels at Bahadurabad

2.4 INSTRUMENTATION

The instrumentation used for the side scan sonar measurements and the subbottom profiling is described in Table 2.4-1 below. Regarding other equipment used at Bahadurabad reference is made to Annex 6.

Survey Application	Type of Instrument	System Description
Side Scan Sonar (SSS) Detection of objects on river bed	EG & G (Edgetech) Model 260	Graphic Recorder Scaled plan view Image Range from 25 m to 600 m to both sides Frequency: 100/500 kHz
Subbottom Profiling (SBP) Sediment Sonar	Datasonics CAP 6600 CHIRP II Operation only in March 1998 for special investigation in front of Test Site II	Reflection Seismic Digital Recording Color Display Frequency: 1to 10 kHz Output Power: 4 kW at two channel

Table 2.4-1: Instrumentation specifications

2.5 WATER LEVEL GAUGES

Water level readings have been taken from two staff gauges, which are located upstream from Section B (gauge A) and downstream from Section H (gauge B). The readings have been scheduled at 8, 13 and 17 hours daily. The exact location and the operating period of the gauges are given in Table 2.5-1. All water level readings refer to PWD.

Gauge No.	Location	E	N	Operation Period
A	Upstream from Section B	471218	778582	Nov. 1995 - Dec. 2000
B	Downstream from Section H	471443	774283	Jul. 1997 - Dec. 2000

Table 2.5-1: Location and operation period of water level gauges

3 FIELD SURVEYS

3.1 BATHYMETRY

The procedure and the hydrographic software used for bathymetric surveys are described in Annex 6.

Three types of bathymetric surveys were defined for the Test Site II.

- Cross Section Survey
  - at the centre of each construction section
  - Length: 200 m
  - surveyed in monsoon 1997 almost every day, later the frequency depended on the river condition.
- Site Survey
  - the area was defined by the river channel in front of the structure, which has been surveyed over a length of 1 km.
  - the line separation was chosen as 20 m.
- Main Channel Survey
  - the area from Harindhara (2.5 km upstream from the structure) to Ghutail (4 km downstream from the structure).
  - maximum survey area: 10 km x 4 km
  - line separation: 100 m

  In 1997 the area was half only, because the monitoring concentrated mainly on the test site area.

3.2 FLOW MEASUREMENTS

The three different methods of flow measurements applied in this Project DGPS Float Tracking, Advanced DGPS Float Tracking and Current Point Measurements are described in Annex 6.

Float tracking was carried out extensively at Bahadurabad Test Site. This method was found appropriate to describe the flow pattern along the structure, above the aprons (during high water stage) and in the main channels in front of the structure. The advanced DGPS float tracking was carried out in 1997 and 1998 only.

Due to the high flow velocities it was difficult to anchor the survey boat in front of the structure carrying out current point measurements by the Valeport current meter. Surface velocities of more than 2 m/s made a proper anchoring even impossible. Therefore, anchor moorings were provided in front of the structure in June 1998. These moorings consisted of heavy blocks (about 1 ton), a wire rope and a drum used as a buoy. It was found practical to keep the survey boat by these moorings over a fixed point for the valeport measurements, but unfortunately floating debris got stuck on the wire ropes and caused the sinking of the buoy.

It was decided to replace the drum buoy by a bigger conical buoy as they are used for navigation on the Jamuna. But the high sedimentation in front of the structure in July 1998 made this idea useless, because the flow velocities decreased significantly.

22

3.3 LAND SURVEYS

The land survey tasks carried out by the electronic total station and the level equipment are similar to the Test Site I at Kamarjani.

The types of surveys were as follows:

- horizontal and vertical control;
- bankline surveys;
- char waterlines, and
- topography (structure, roads, objects, etc.)

3.4 SIDE SCAN SONAR AND SUBBOTTOM SURVEYS

3.4.1 Side Scan Sonar Description

A Side Scan Sonar (SSS) survey provides a plan view map of the riverbed. The sonar is scanning the river bottom by emitting pulses in a thin, fanshaped pattern that spreads downward to either side of the towed fish in a plane perpendicular to its path. It derives its information from reflected acoustic energy.

Depending on the backscattered strength of the signal the recorder prints an image graphic of 16 different grey tones (sonograph). Good acoustic reflectors like cc-blocks, boulders or sand ripples are represented by darkened areas on the record. Depressions or other features scanned from the acoustic beam are indicated by light areas. The high system resolution (pixel size of 1/8 mm) enables to recognise single boulders, blocks or other objects on the recorded image. Objects can be positioned on the sonograph like on a scaled map (scale depends on selected range). The image characteristics are a function of both bottom topography and sediment property variation.

The records from the model EG&G 260 as it has been used in this Project do not have scale distortions as microprocessors in the signal processor apply an automatic amplitude correction to the received data such that they relate directly and consistently to the back scattering characteristics of the riverbed. True horizontal scale is represented, as derived from fish height and angular slant range to the bottom targets. Ship speed is used to correct sonar data for along-track compressional distortion.

Fig. 3.4-1: Side scan sonar towfish

3.4.2 Subbottom Profiler Description

The subbottom profiler (SBP) generally uses the seismic reflection method. A seismic pulse is generated (at a source) and transmitted into the water column. On the riverbed and at the subbottom layers the energy is reflected and received by a special receiver. After amplification the data are displayed on a graphic recorder or monitor.

Point reflectors, like single boulders or other targets are characterised by hyperbolic reflections. These hyperbolae are side lobes generated by the changing distance between transmitter and target.

A relatively low frequency (2 to 7 kHz) was used in order to acquire substantial subbottom penetration into the sediments below the rive bed. The system was internally triggered at a repetition rate varying between 4 shots per second (250 ms) and 8 shots per second (125 ms). Data were recorded at a rate of 1024 or 2048 samples per shot. Therefore, the resolution of digital data varied between 0.12 ms and 0.06 ms. At a speed of sound of 1500 m/s the resolution varies between 4.5 cm and 9 cm.

3.4.3 System Configuration aboard Survey Boat

The survey boat 'Obelix' has been used to carry out the Side Scan Sonar and Subbottom Surveys alternatively. For navigation and positioning the same setup as for bathymetric surveys was used. The system configuration is shown in Fig. 3.4-2. The Masterchart (Hydrographic Software) data logging format has been chosen as follows:

- time;
- fix number;
- Easting;
- Northing;
- depth;
- speed made good (SMG);
- distance along line (DAL), and
- distance offline (DOL).

Every second one data string is logged in the navigation file. The side scan sonar recorder provides only analog records. It is not equipped with a navigation interface board. Therefore the side scan sonar records were annotated manually using the same fix numbers as generated by the Masterchart navigation. A fix interval of 10 seconds had been chosen. That means the side scan sonar records were provided every 15m with a position fix. The subbottom profile could be interfaced with the Masterchart navigation, i.e. the digital seismic data could automatically be provided with navigation data.

The side scan sonar towfish was mounted 1m starboard front side of the survey boat, and could be lowered by the anchor winch. The subbottom transducer was mounted at the portside of the survey boat well in front of the wake in order to reduce ships noise interference as much as possible. The transducer depth was 0.45 m below the water level. The offsets of the towfish and the transducer in relation to the GPS antenna were corrected by the Masterchart.

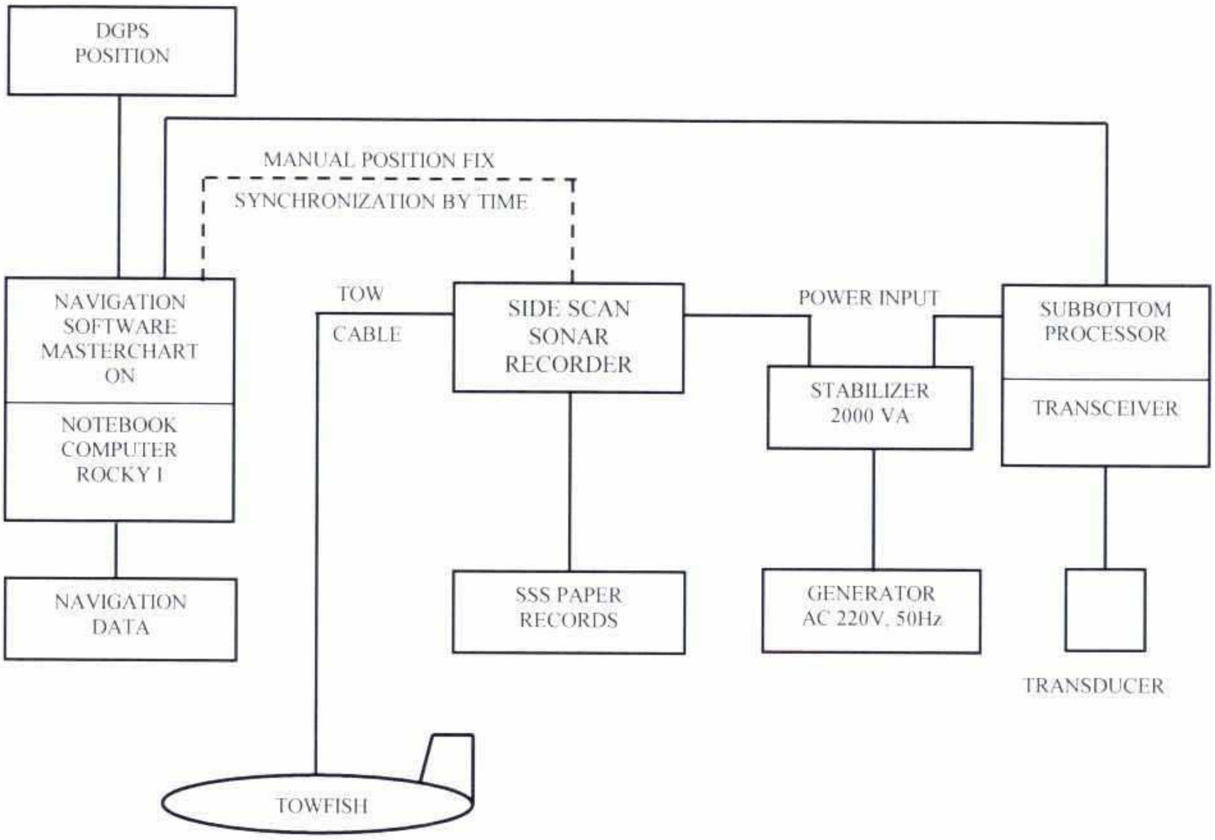


Fig. 3.4-2: System configuration aboard survey boat Obelix for SSS and SBP surveys

4 THE REVETMENT TEST STRUCTURE

4.1 SUMMARY

A first reconnaissance bathymetry survey in the area of Bahadurabad was carried out in June 1995 (Fig. 4.1-1). The bank was about 400 m off the original proposed structure location of 1995. Water depths of more than 20 m were surveyed in front of Bahadurabad Ghat.

Because the test structure could not be finished at the original location during the dry season 1995/96, the final layout of the revetment structure was located about 120 m further to the hinterland (Fig. 4.1-2). The execution of works started in November 1996 and the structure was completed in all respects on June 12, 1997.

This chapter describes the bathymetry and flow conditions at the test site area during three years of monitoring from monsoon 1997 to monsoon 1999. The test site monitoring is summarized in the following, whereas the bathymetry and flow conditions in front of the structure are described more in detail in Chapter 4.2 to 4.4.

For the monitoring of the main channels in front of the test site area reference is made to Annex 1.

Already during the first two months after completion all falling aprons along Section B to H-1 started to function. Along Section C to H-1 the falling aprons launched to a slope inclination of 1V:2H as it was designed. The slope inclination at Section B was found as 1V:3H.

A part of the launching aprons at Section C started to launch already mid June 1997 and this process continued during the monsoon 1998. The launching aprons in the other sections remained stable and no movement/launching was until the end of the monitoring period.

After launching of the falling aprons the river channel in front of the structure started to scour its bed by about 4 m. High surface flow velocities between 3 to 4 m/s were surveyed in the channel in front of the structure.

The surface flow velocity above the falling aprons was reduced to 2 to 3 m/s and above the launching aprons to 0.5 to 1.5 m/s were measured in the months of June and July 1997. In 6 m depth the flow velocity above the falling aprons was 1 to 2 m/s.

In August 1997 severe scouring started at the downstream termination of the structure.

Downstream from Section H an embayment developed during the monsoon 1997. A maximum scour hole below -14 m+PWD was surveyed 60 m in front of the transition between Section G and Section H. The bank just downstream from Section H was eroded over a distance of 120 m from June to September 1997.

The structure was attacked again in June 1998 but sedimentation started in front of the structure during July the main channel bend shifted further downstream,. On August 09 the bed level was at 15 to 17 m+PWD, which was far higher than SLW (13.3 m+PWD). However, the riverbed was eroded again at the end of 1998 and in June 1999 Section G, H-1 and H-2 came again under attack. A scour hole below -7 m+PWD developed temporarily downstream from Section H, but no damages to the structure and no further bank erosion was observed. From July 1999 the area silted up and the test structure was not under attack until end of 1999.

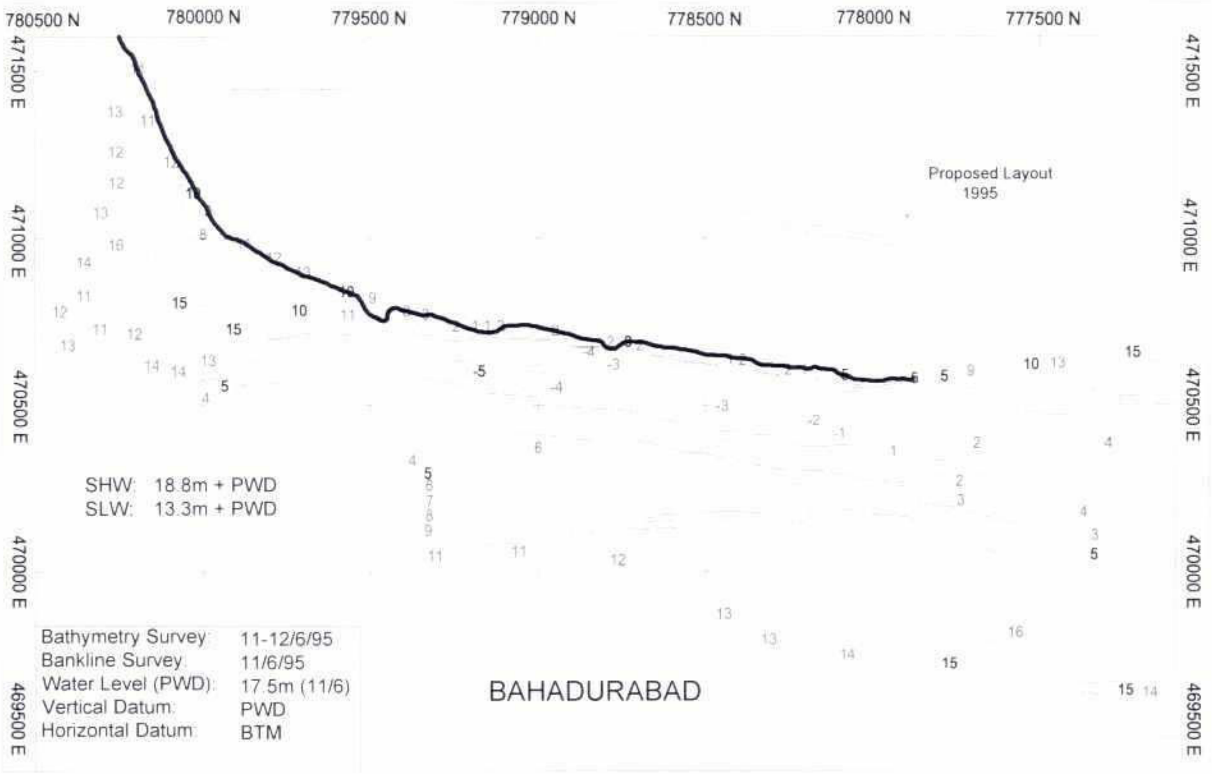


Fig. 4.1-1: Bathymetry in June 1995

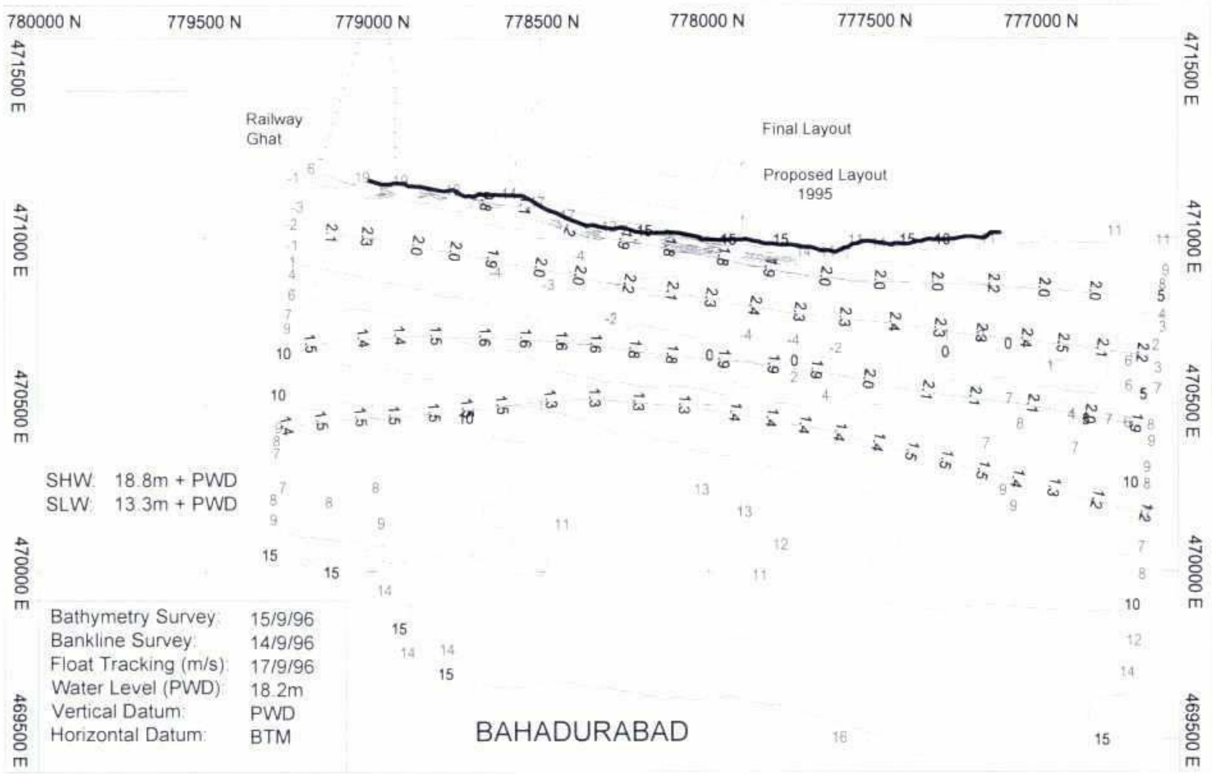


Fig. 4.1-2: Bathymetry in September 1996

4.2 BATHYMETRY AND FLOW CONDITIONS

The following Fig. 4.2-1 to Fig. 4.2-22 present the bathymetry and flow conditions in front of the test structure from May 1997 to July 1999. The charts are based on the monitoring site surveys. Fig. 4.2-7 shows the progressive bank erosion downstream from the structure from May to September 1997. The presented flow lines and flow velocities are derived from the DGPS float tracking surveys. A description of the chronological hydrographic conditions is summarized already in the previous chapter. Details of interest will be described in the following.

At the high water stage in July 1997 the survey boat could pass the falling aprons to cover the launching aprons with bathymetry and flow information as well (Fig. 4.2-4). The flow over the launching apron entered at the transition of Section B and Section C and moved out at the downstream termination. Along the structure the flow velocity accelerated from 0.5 m/s to a maximum of 1.6 m/s. This flow description over the aprons was a general observation during the monsoon 1997 as it was reported in the logbook.

From the survey of July 12 (Fig. 4.2-4) it can be recognised that the main attack on the structure started at the transition of Section B and Section C, where the maximum launching of the aprons took place. The complete areas of the falling aprons at Section D and E were already slided to the channel to develop a slope of an inclination 1V:2H. Since the areas of the falling aprons at Section F and G are larger than at the upstream sections, the cc-blocks only partly slid to the river. The slope inclination is recorded in these sections as 1V:2H as well whereas the slope inclination in Section B is recorded as 1V:3H.

In the first week of August 1997 the flow over the launching apron shifted to the channel at the transition of Section E and F. This might be explained by the scour hole in front of the structure at that place, which attracted the flow (Fig. 4.2-5). Later the flow changed again at the downstream termination as before since the scour hole shifted further downstream.

On September 25 the water level reached the last peak of 1997 (19.1 m+PWD), which was higher than the water level peak of August. A survey from that day is presented in Fig. 4.2-8. The areas over the aprons could be monitored once again. The flow track passed the same way over the aprons as in July but with reduced velocities. Especially along Section C low velocities were found over the launching apron. That resulted in sedimentation in that area. The river bed morphology of that survey shows significant changes downstream from Section H. The erosion process started already in that area on August 10. Strong eddies and backflow along the bank were observed from that time in that area. This resulted in big scouring and further slides of cc-blocks and boulders, which were placed at the falling aprons of Section G and H.

The bankline surveys downstream from the structure carried out from May to September 1997 are presented in Fig. 4.2-7. The attack on the downstream part of the structure continued in June 1998 (Fig. 4.2-12).

The highest water level at Bahadurabad during the total monitoring period was recorded on September 07 and 08, 1998 at 20.12 m+PWD. Two days later a survey of the test site was carried out (Fig. 4.2-15). Although the river channel was silted up in the previous two months (Fig. 4.2-13 and Fig. 4.2-14), high flow velocities of about 1.5 m/s were recorded over the launching apron.

In June 1999 less flow velocities were measured in front of the structure (Fig. 4.2-20). But downstream from Section H the backflow along the bank and the eddies were present again in that area, which was surveyed on June 28 (Fig. 4.2-21).

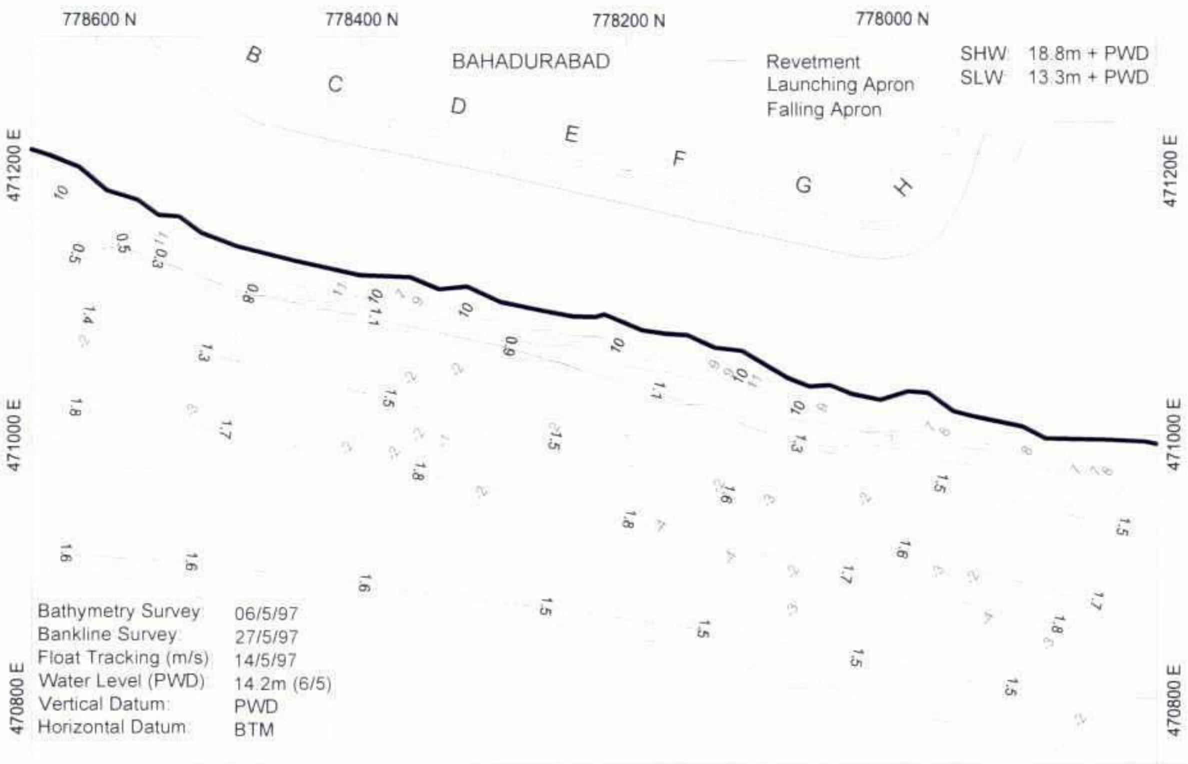


Fig. 4.2-1: Bathymetry and flow in May 1997

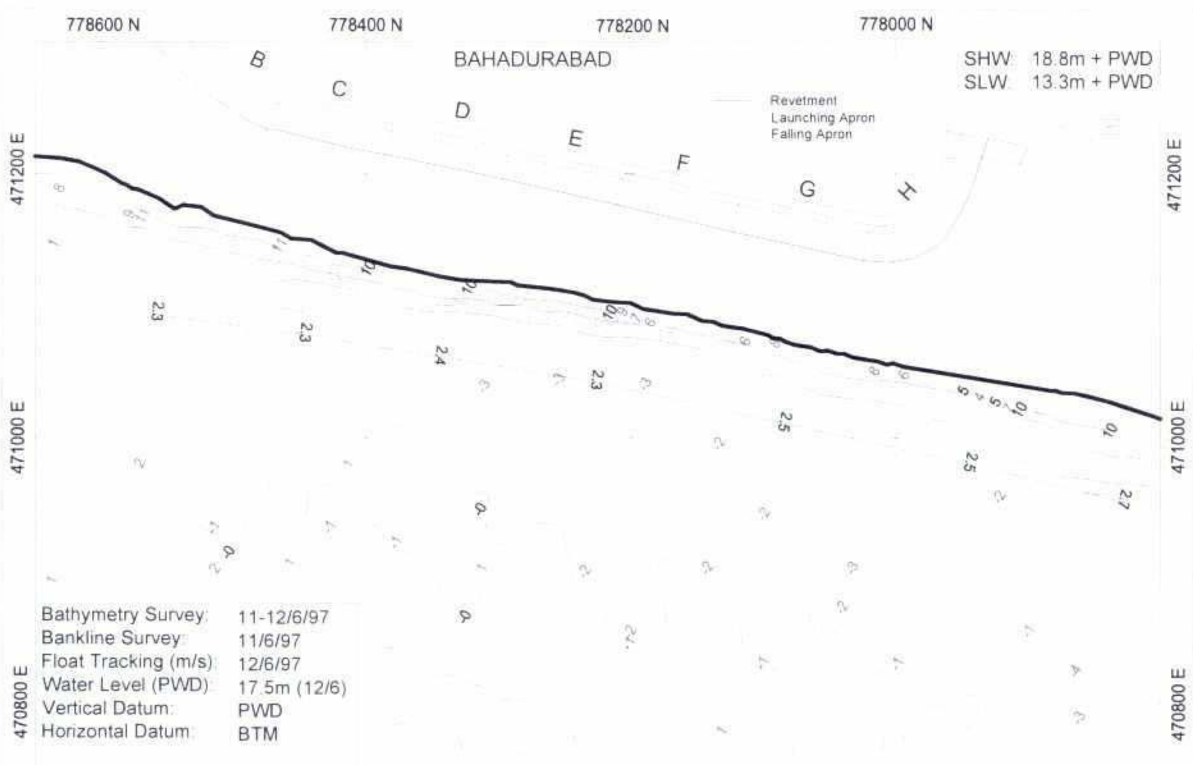


Fig. 4.2-2: Bathymetry and flow in June 1997

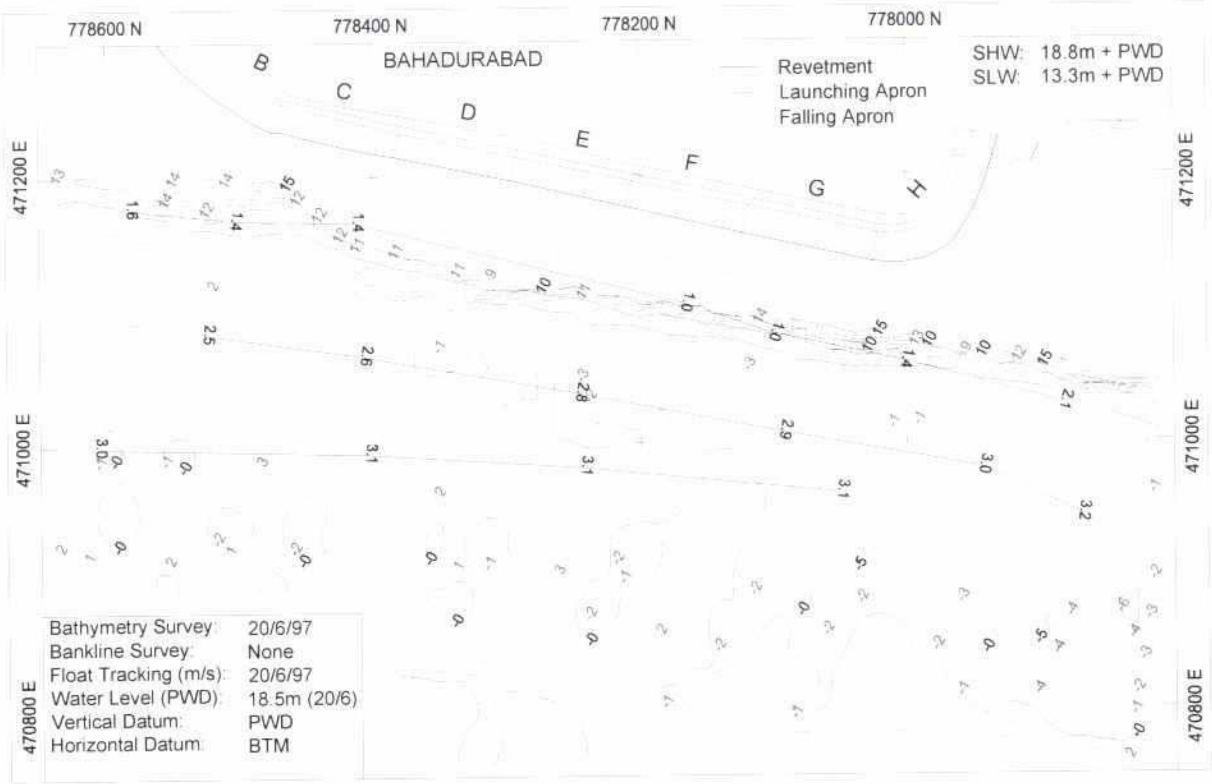


Fig. 4.2-3: Bathymetry and flow in June 1997

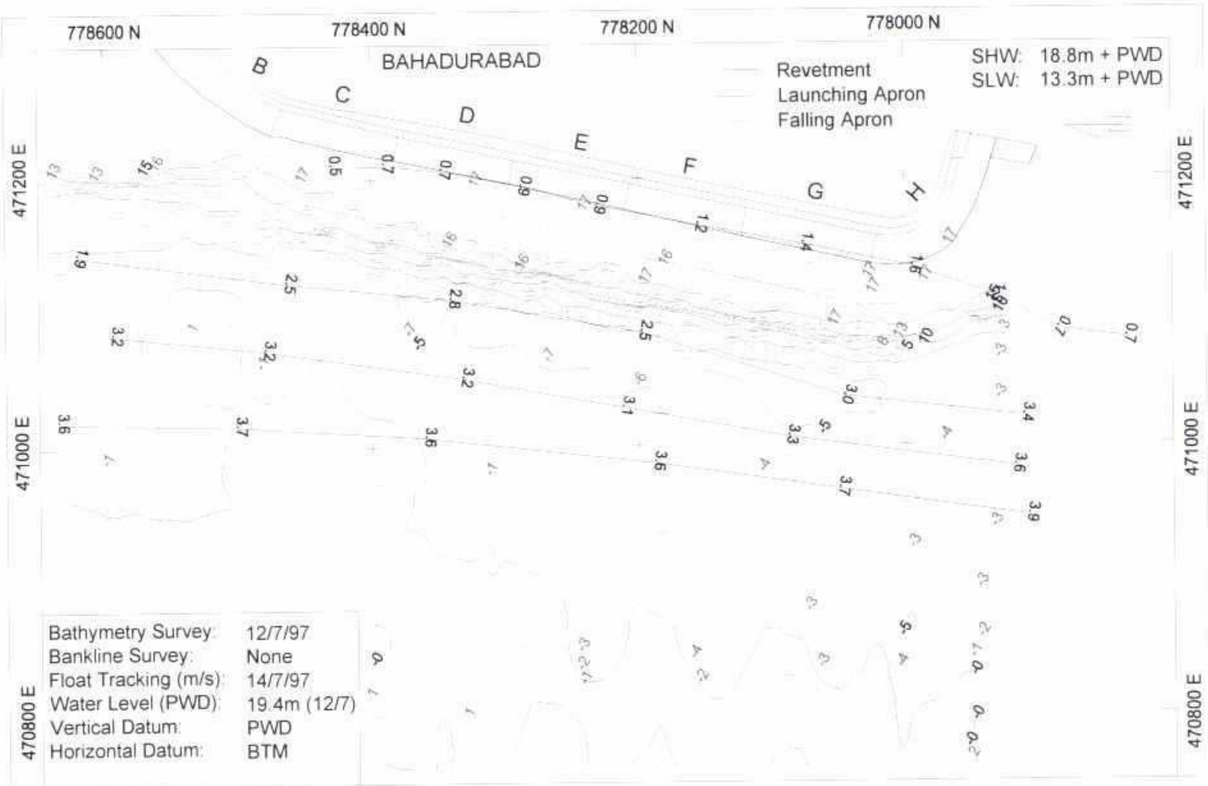


Fig. 4.2-4: Bathymetry and flow in July 1997

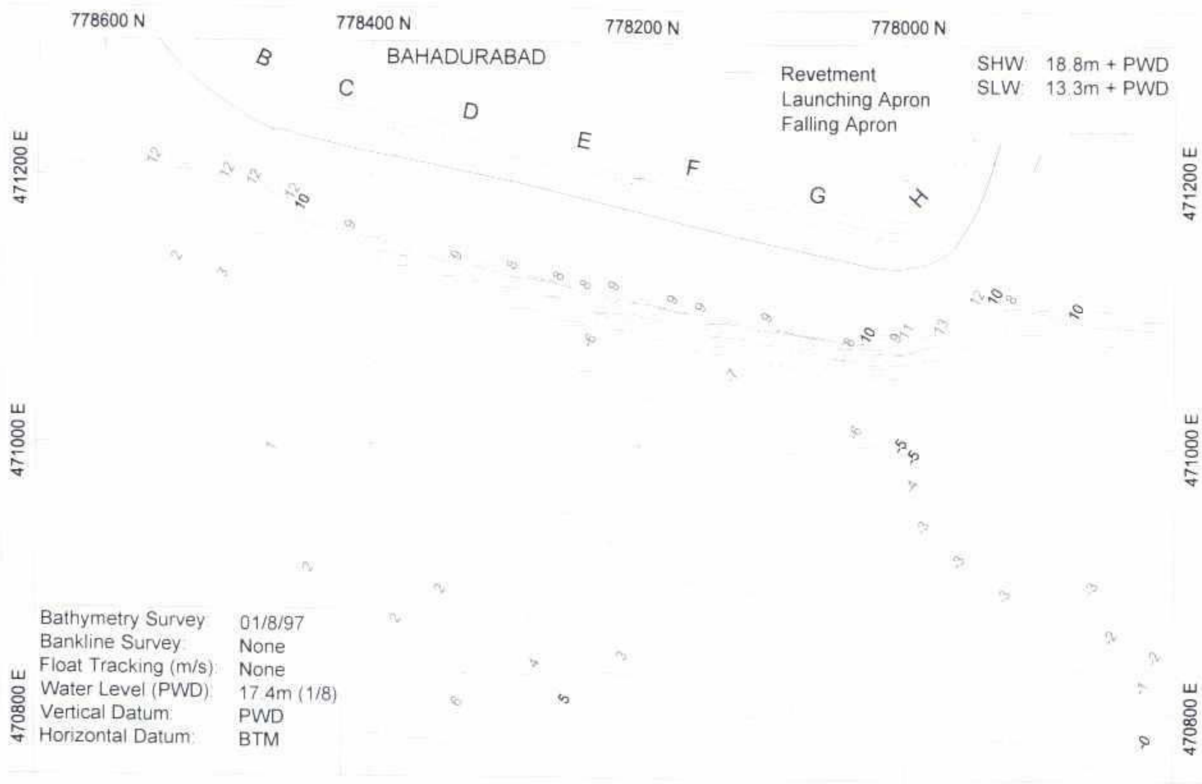


Fig. 4.2-5: Bathymetry in August 1997

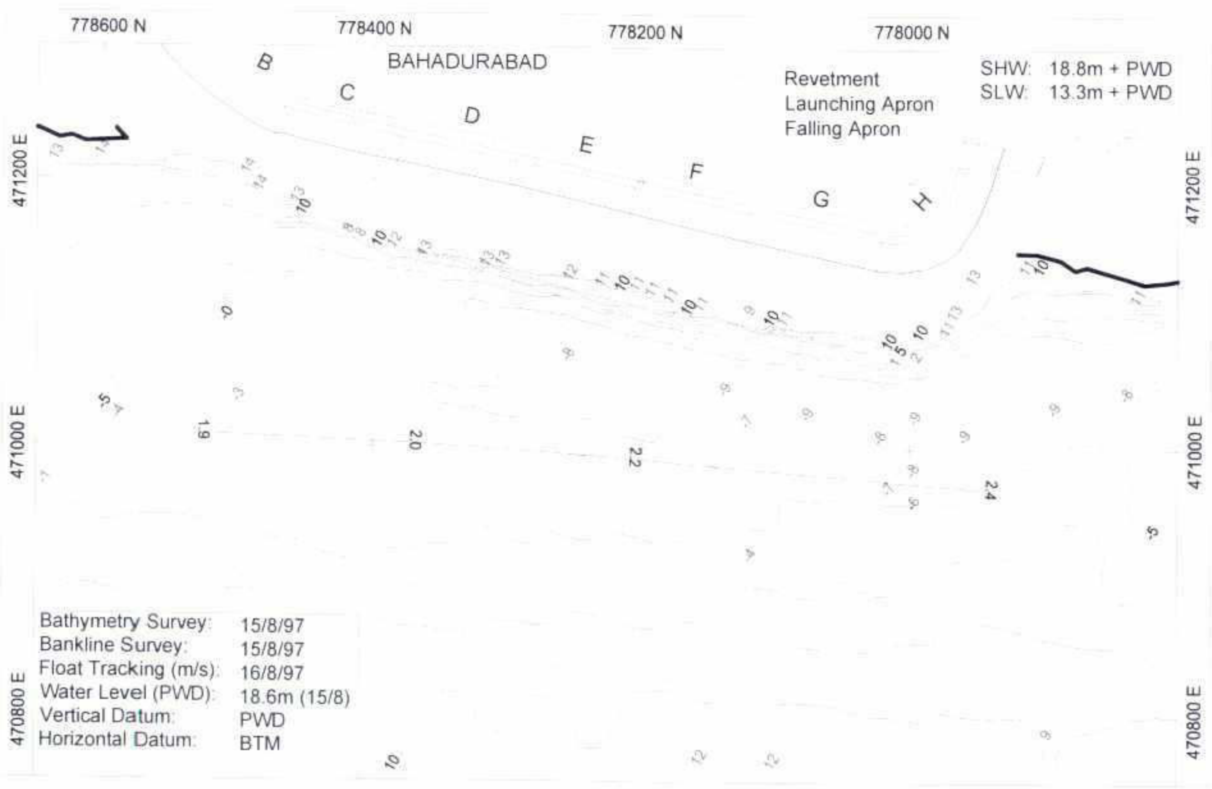


Fig. 4.2-6: Bathymetry and flow in August 1997

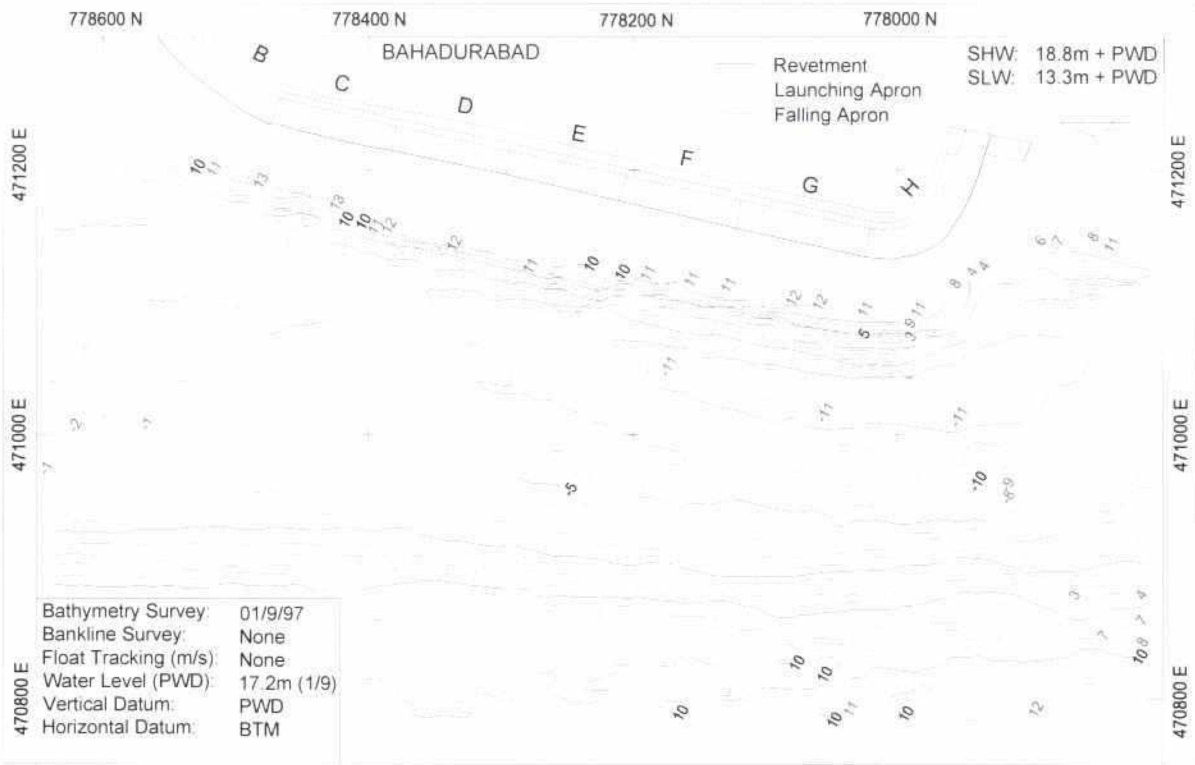


Fig. 4.2-7: Bathymetry in September 1997

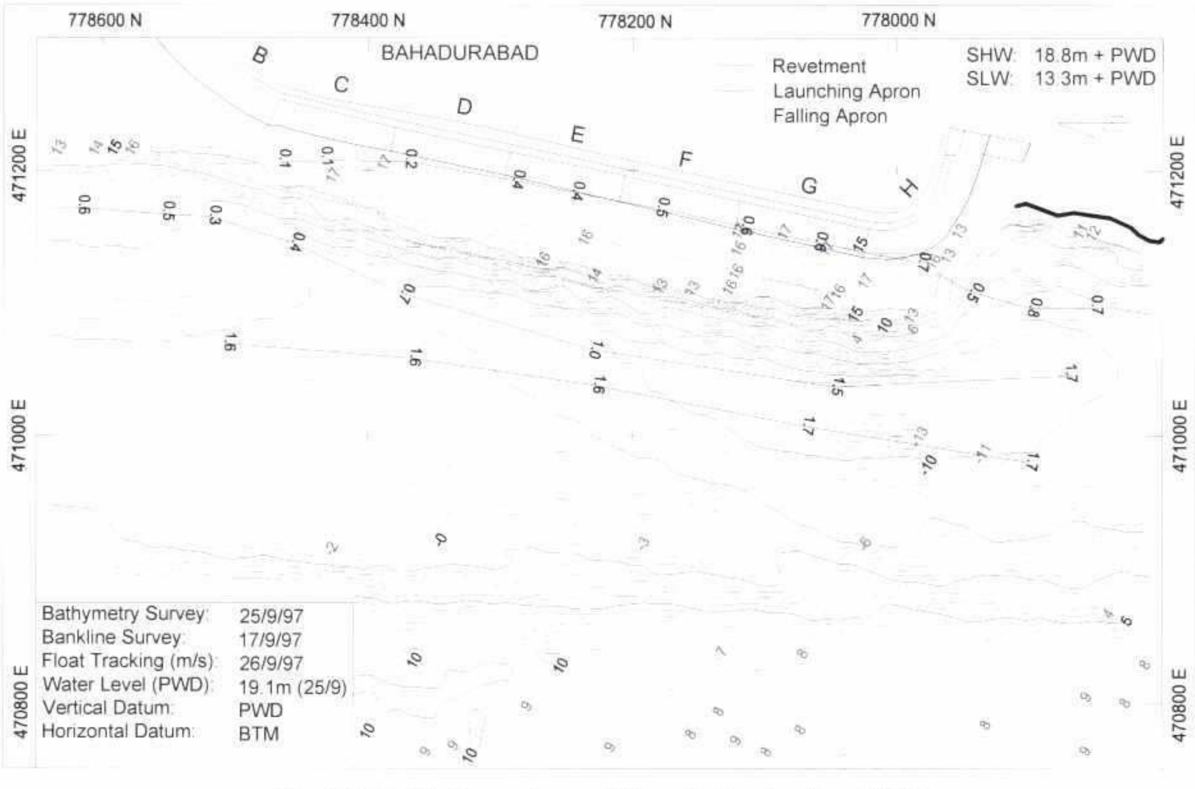


Fig. 4.2-8: Bathymetry and flow in September 1997

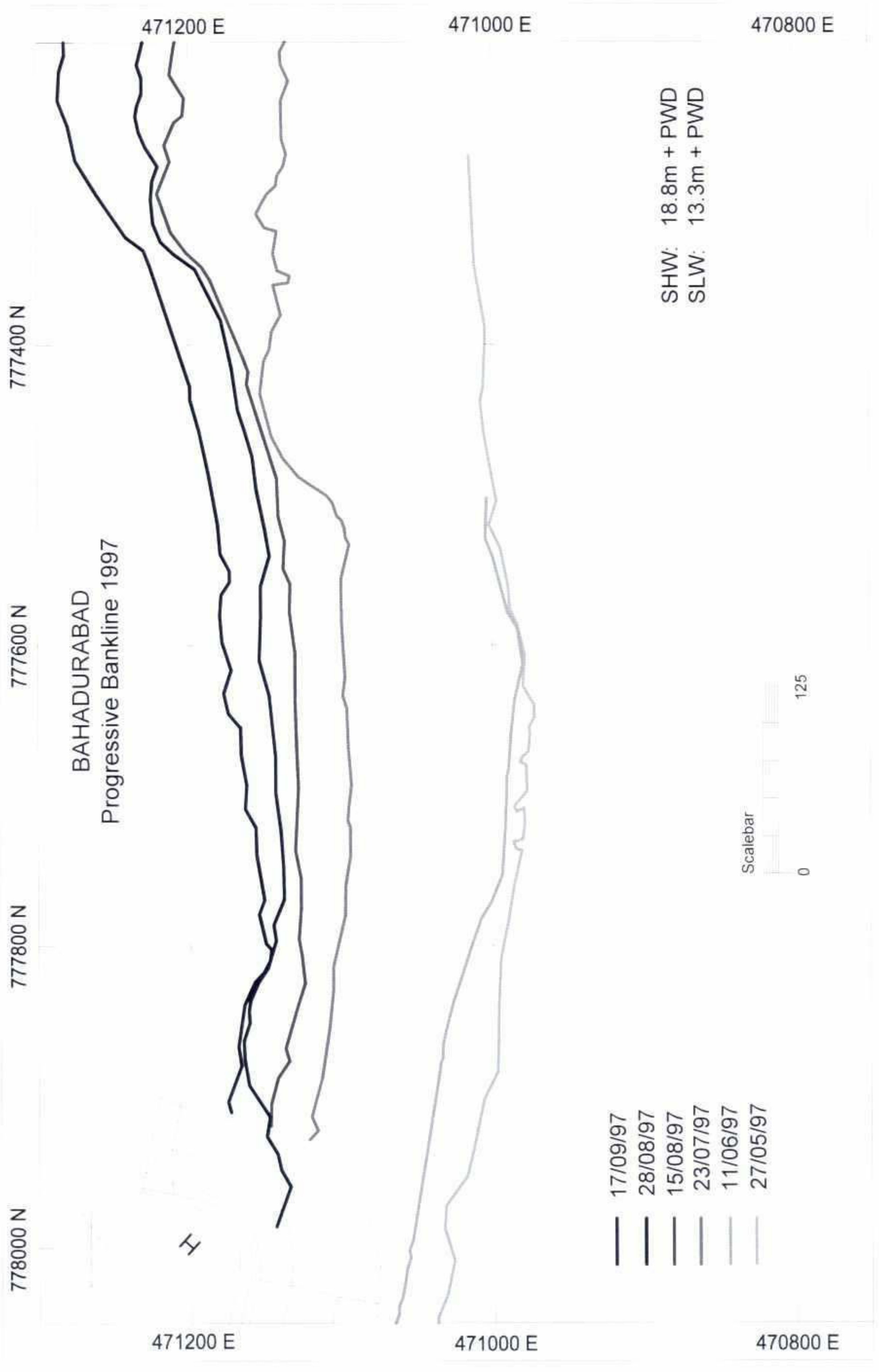


Fig. 4.2-9: Progressive bankline downstream from Test Site in

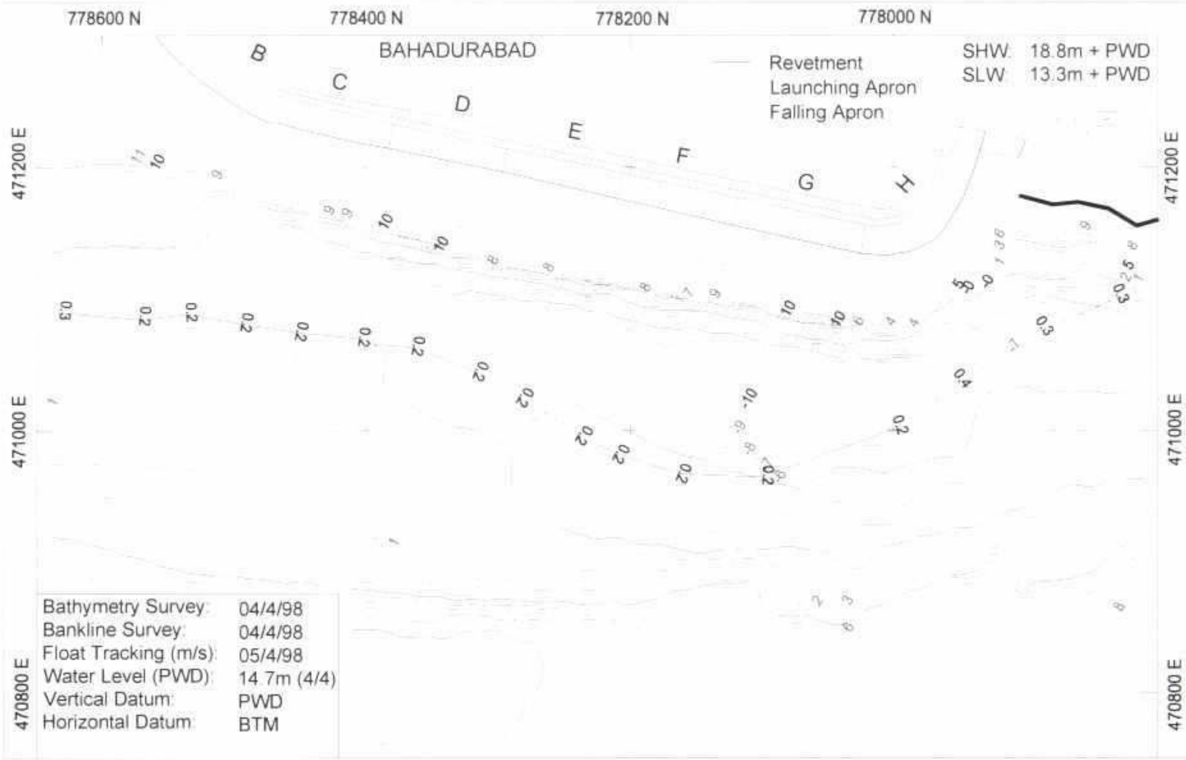


Fig. 4.2-10: Bathymetry and flow in April 1998

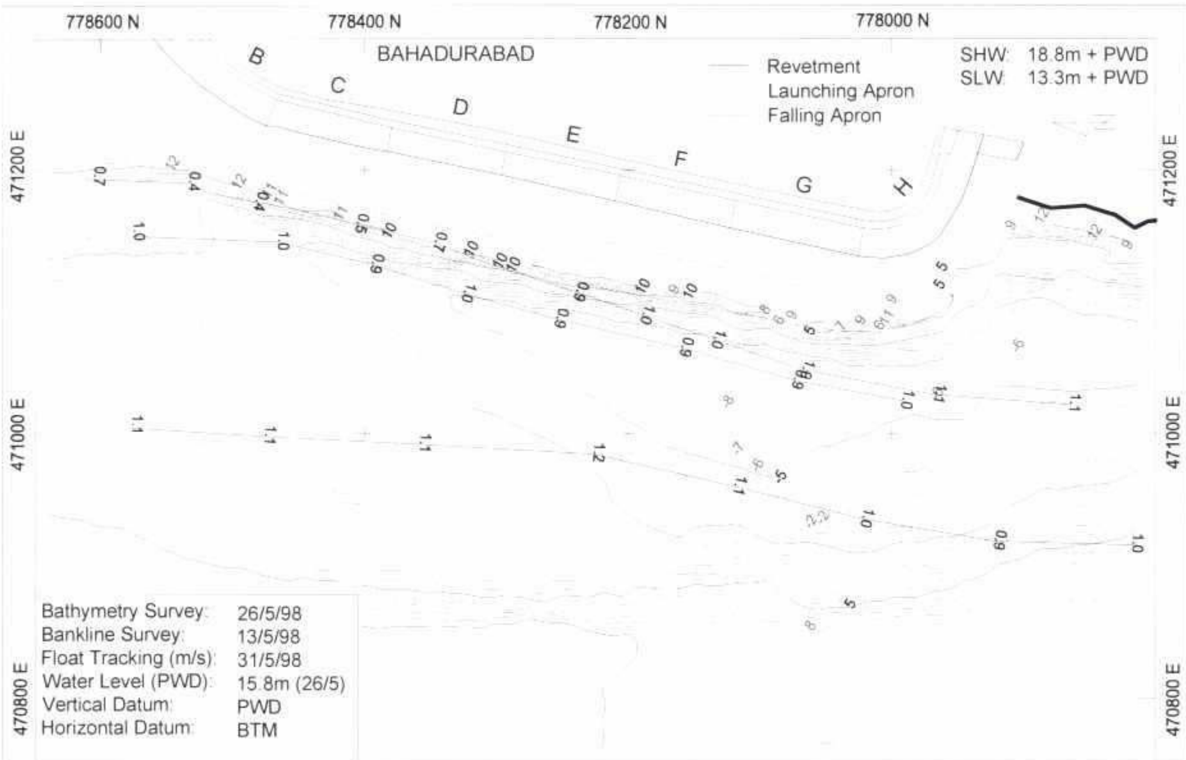


Fig. 4.2-11: Bathymetry and flow in May 1998

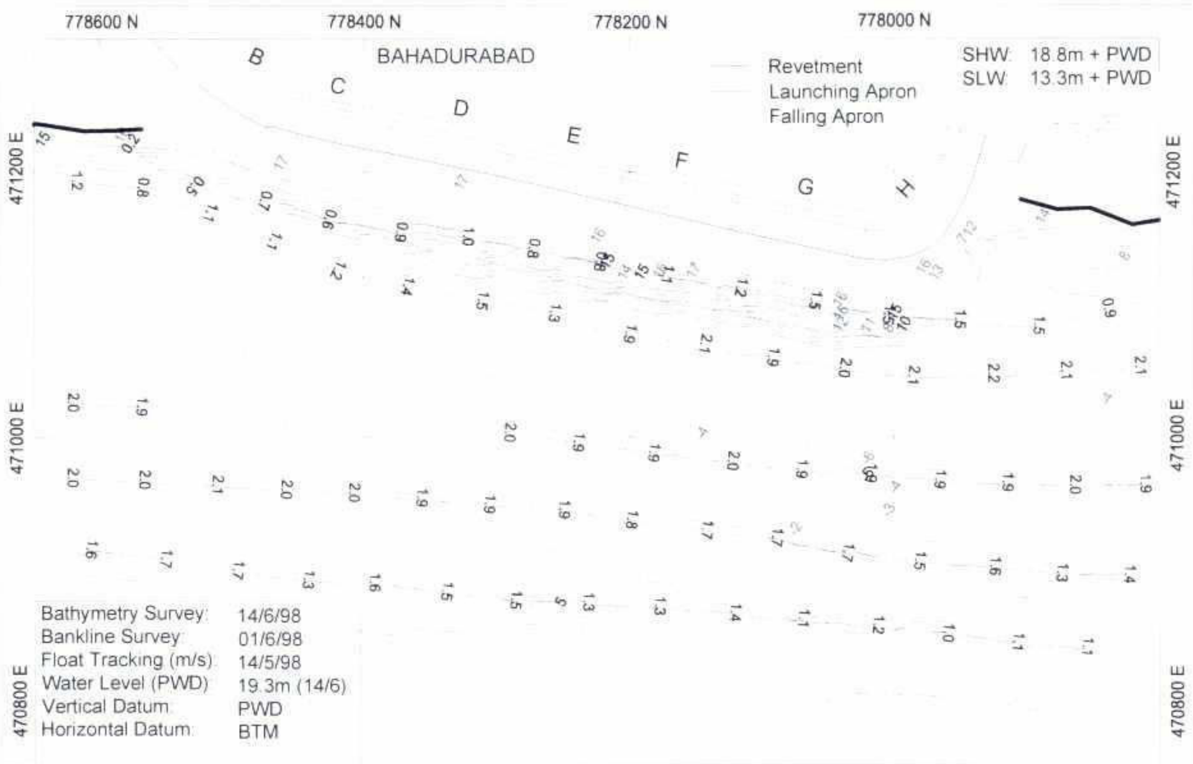


Fig. 4.2-12: Bathymetry and flow in June 1998

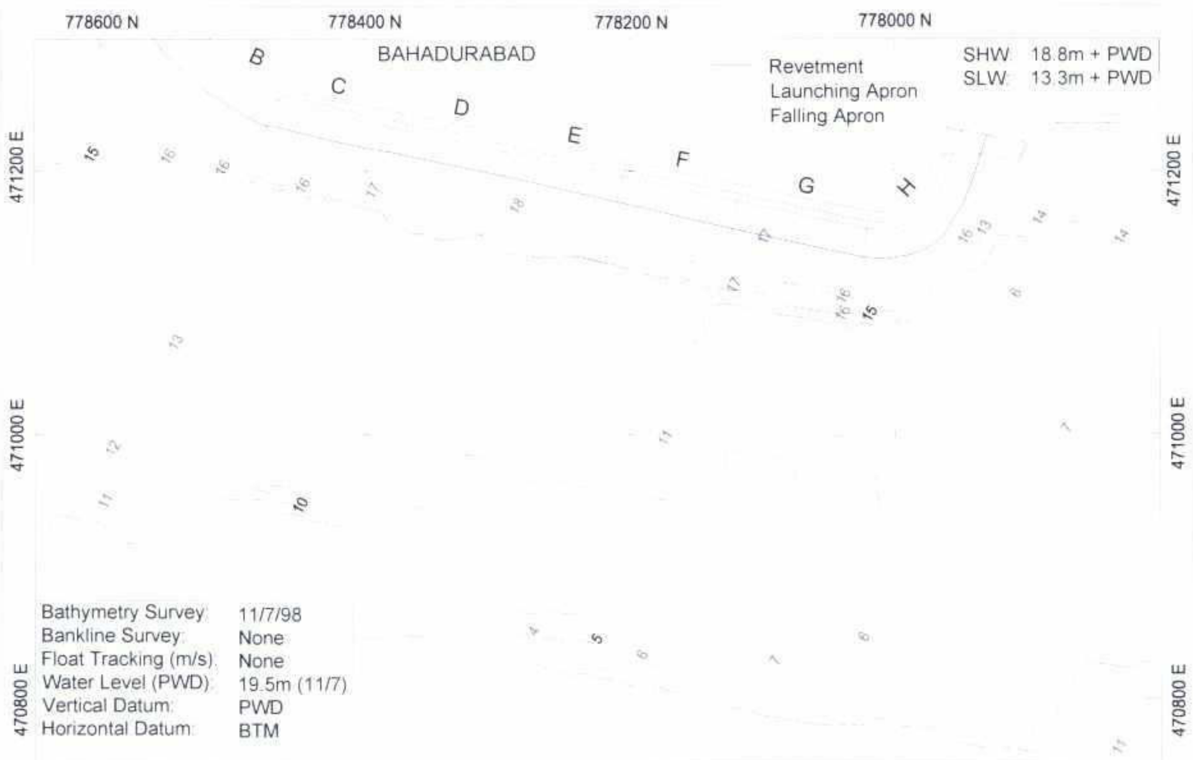


Fig. 4.2-13: Bathymetry in July 1998

108

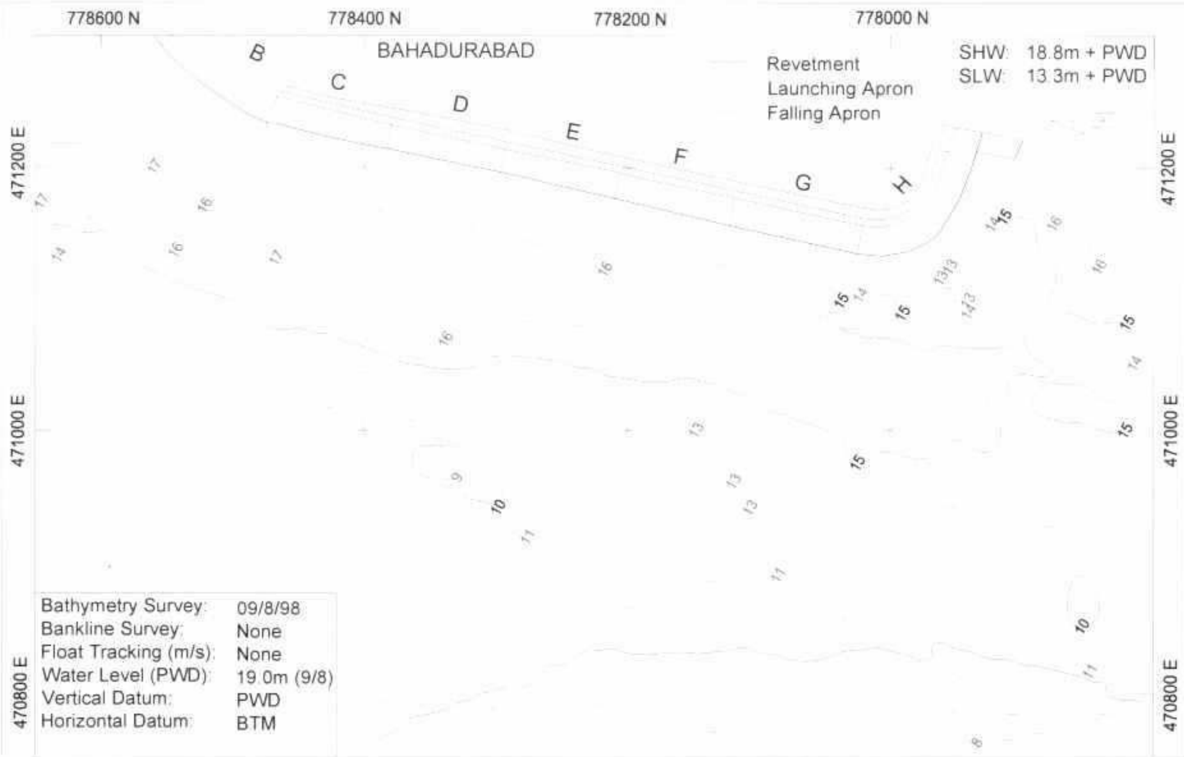


Fig. 4.2-14: Bathymetry in August 1998

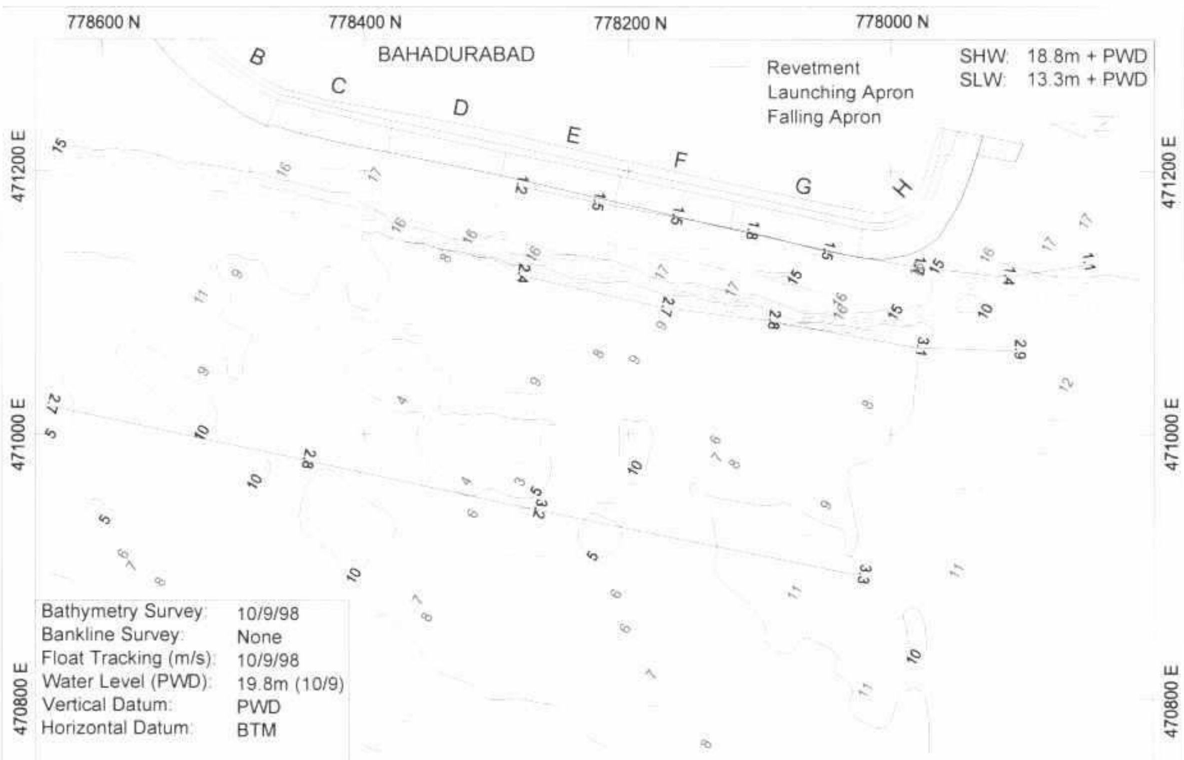


Fig. 4.2-15: Bathymetry in September 1998

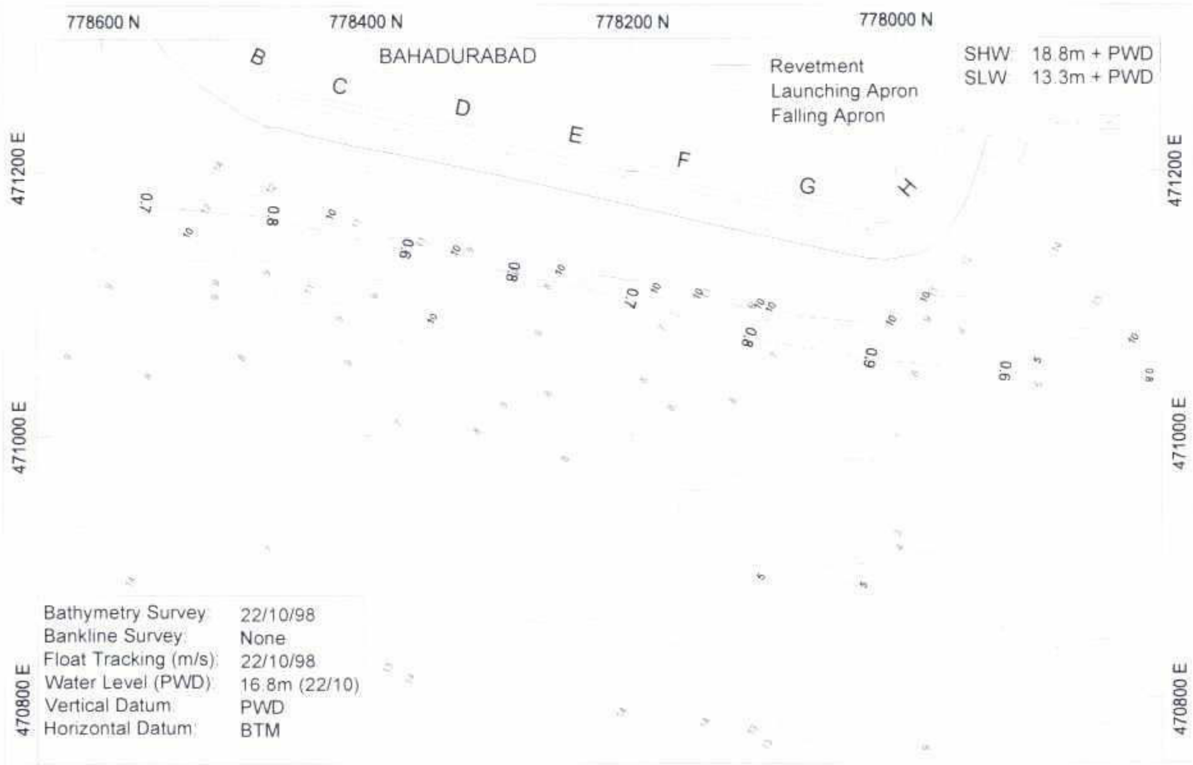


Fig. 4.2-16: Bathymetry and flow in October 1998

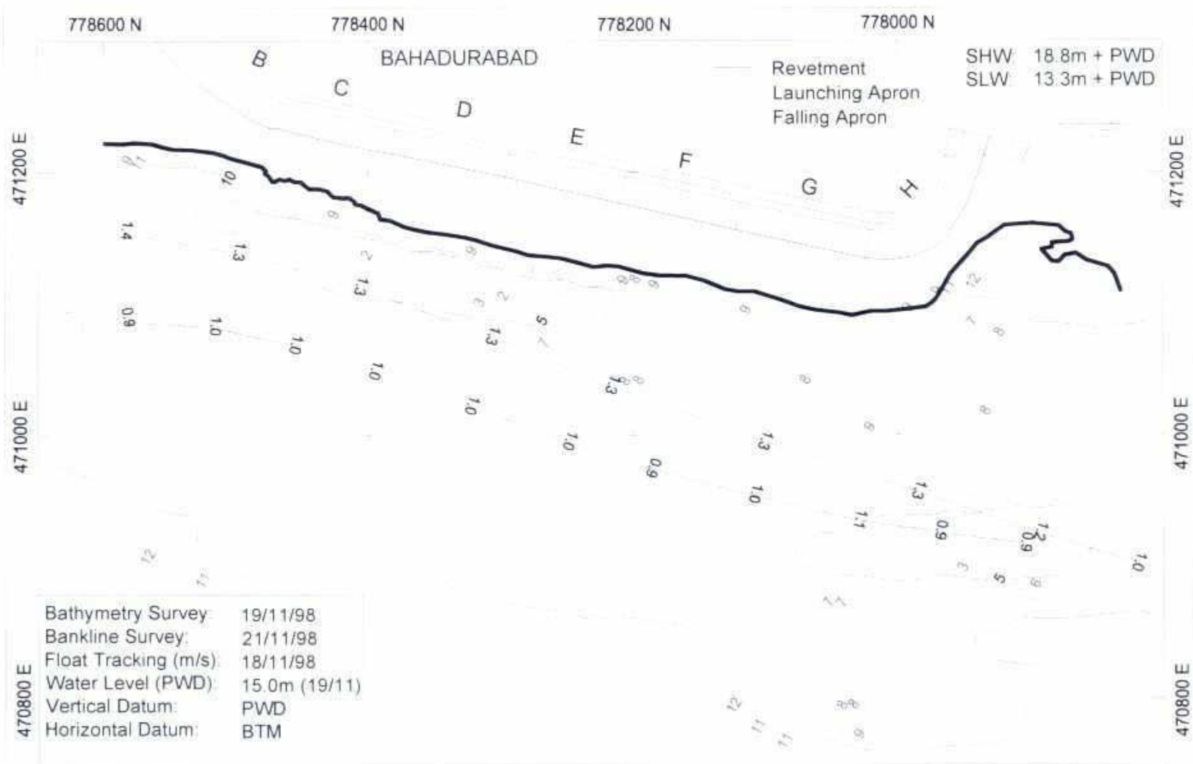


Fig. 4.2-17: Bathymetry and flow in November 1998

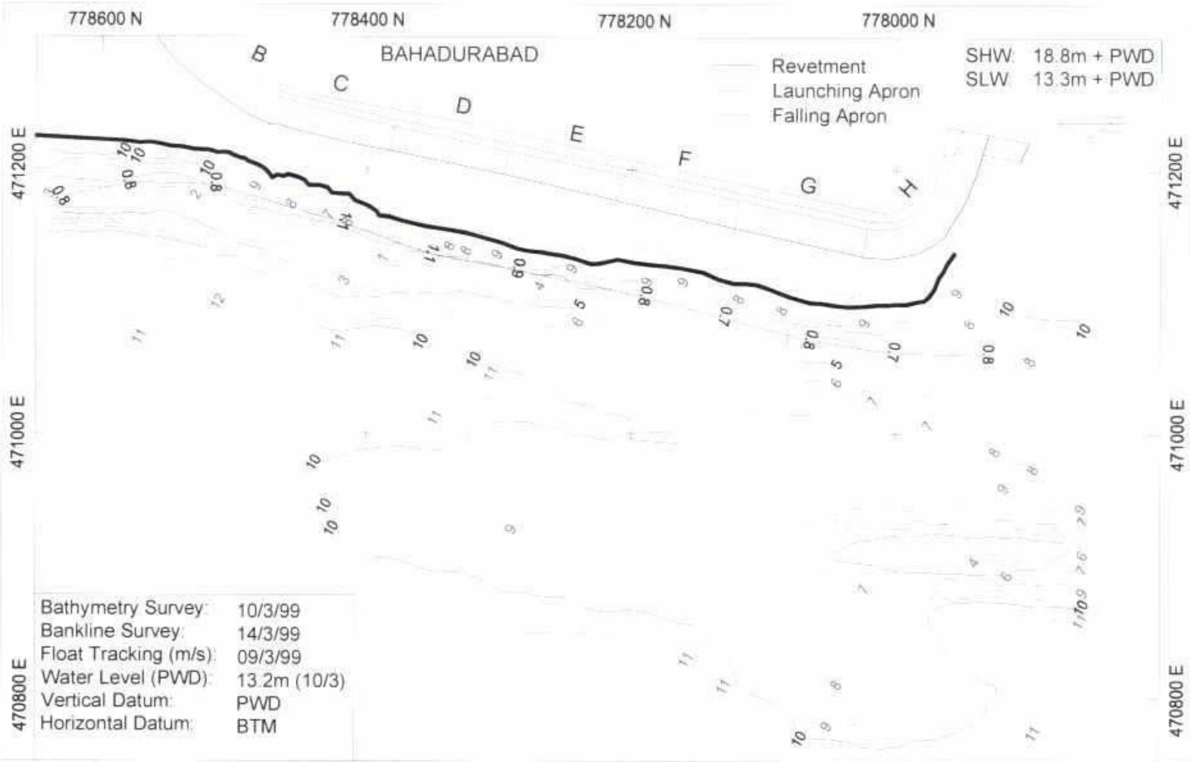


Fig. 4.2-18: Bathymetry and flow in March 1998

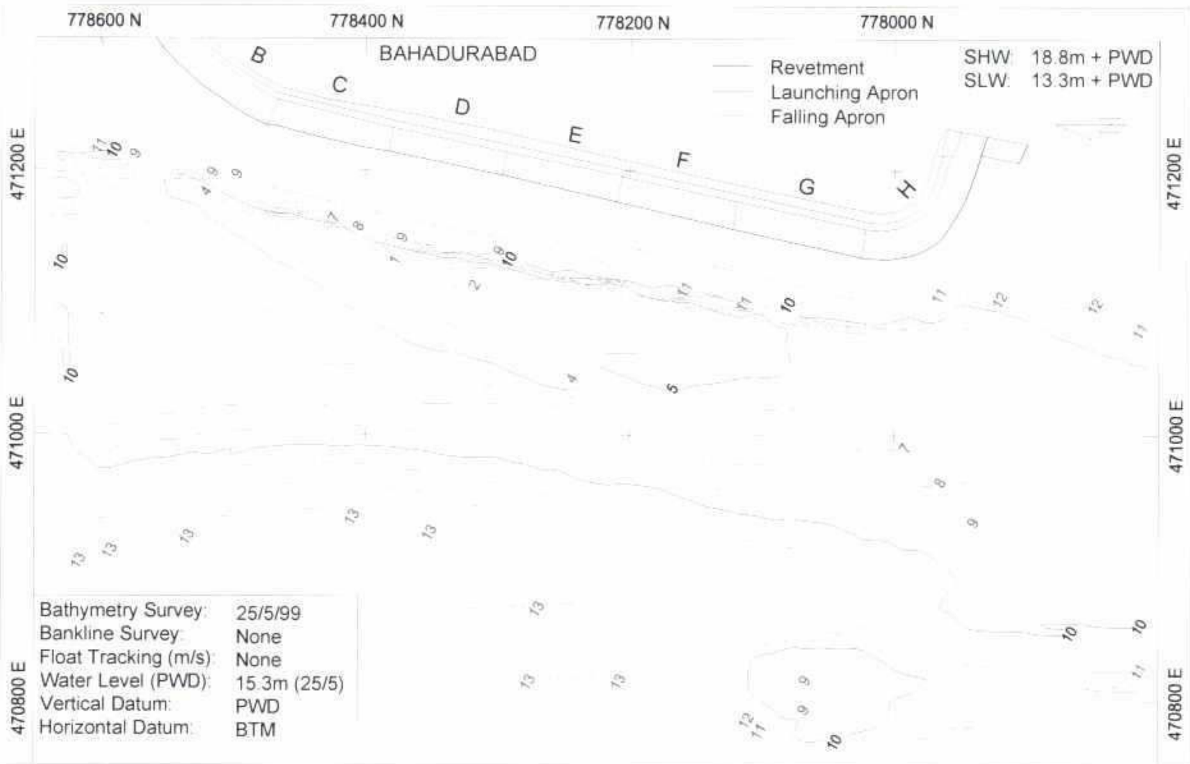


Fig. 4.2-19: Bathymetry in May 1999

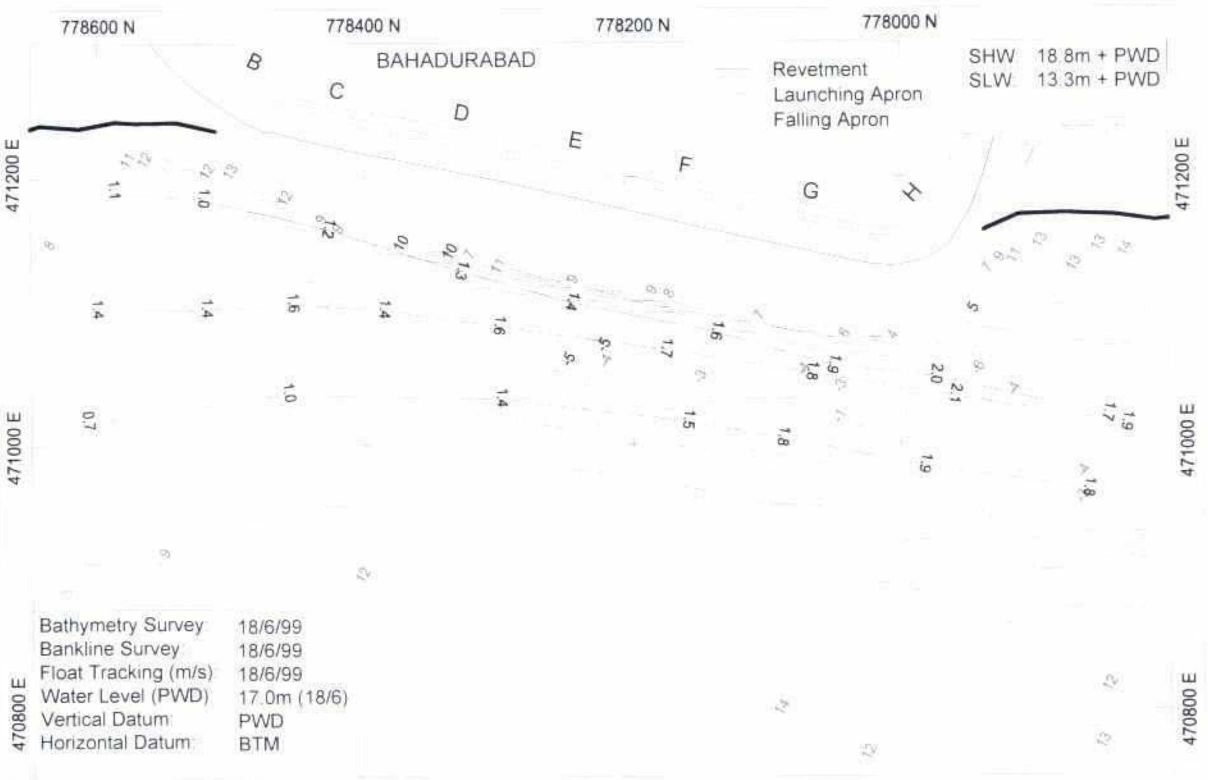


Fig. 4.2-20: Bathymetry and flow in June 1999

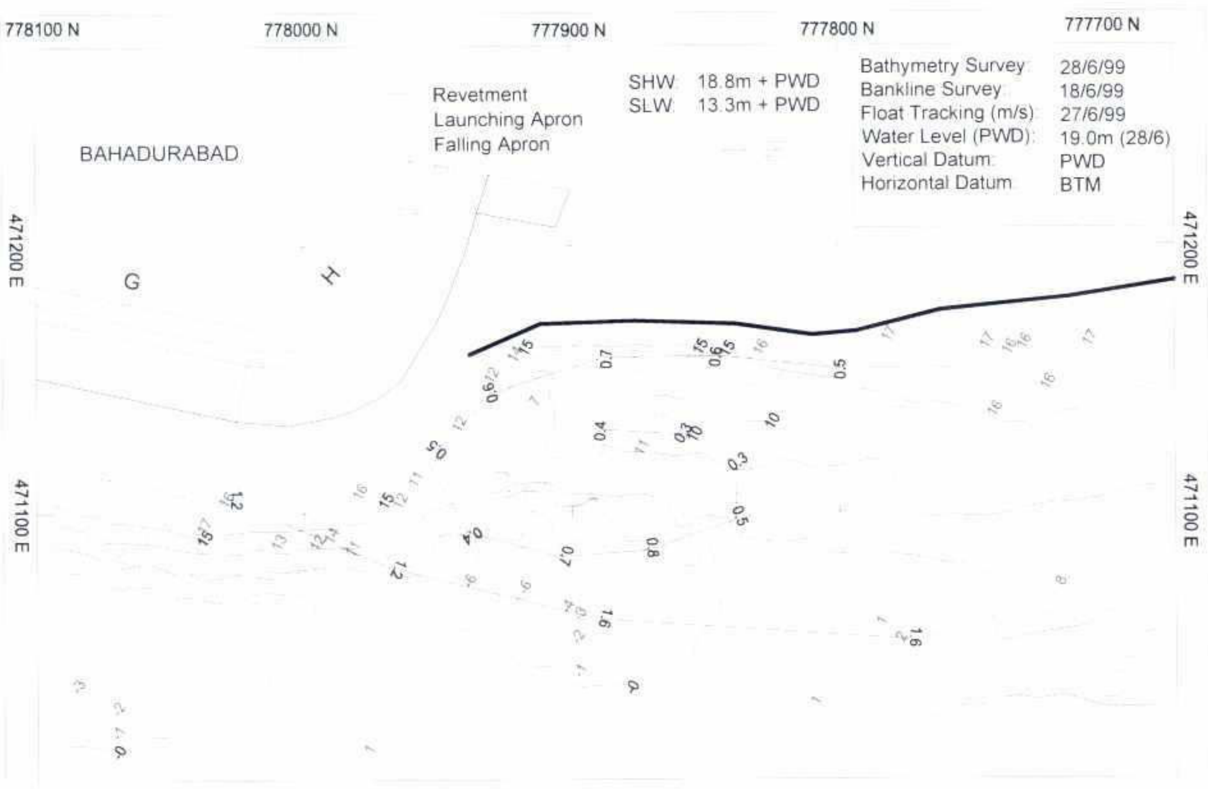


Fig. 4.2-21: Bathymetry and flow downstream from the Test Site in June 1999

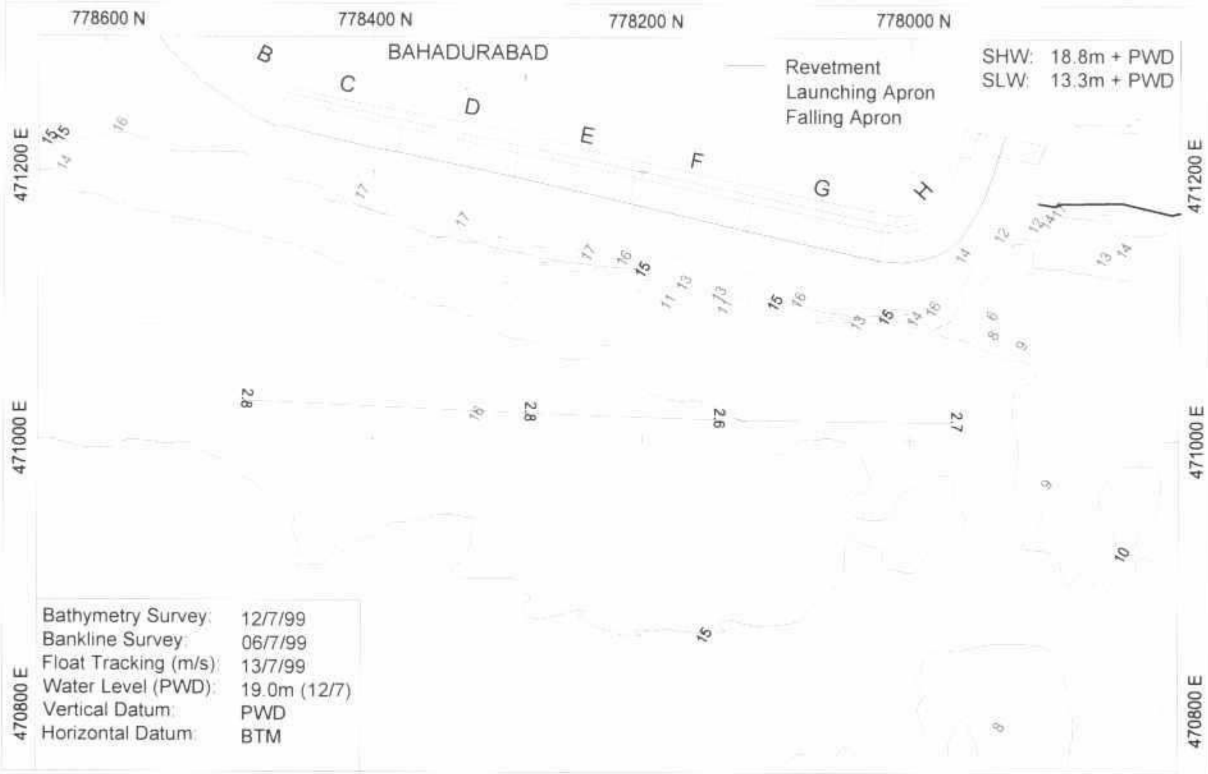
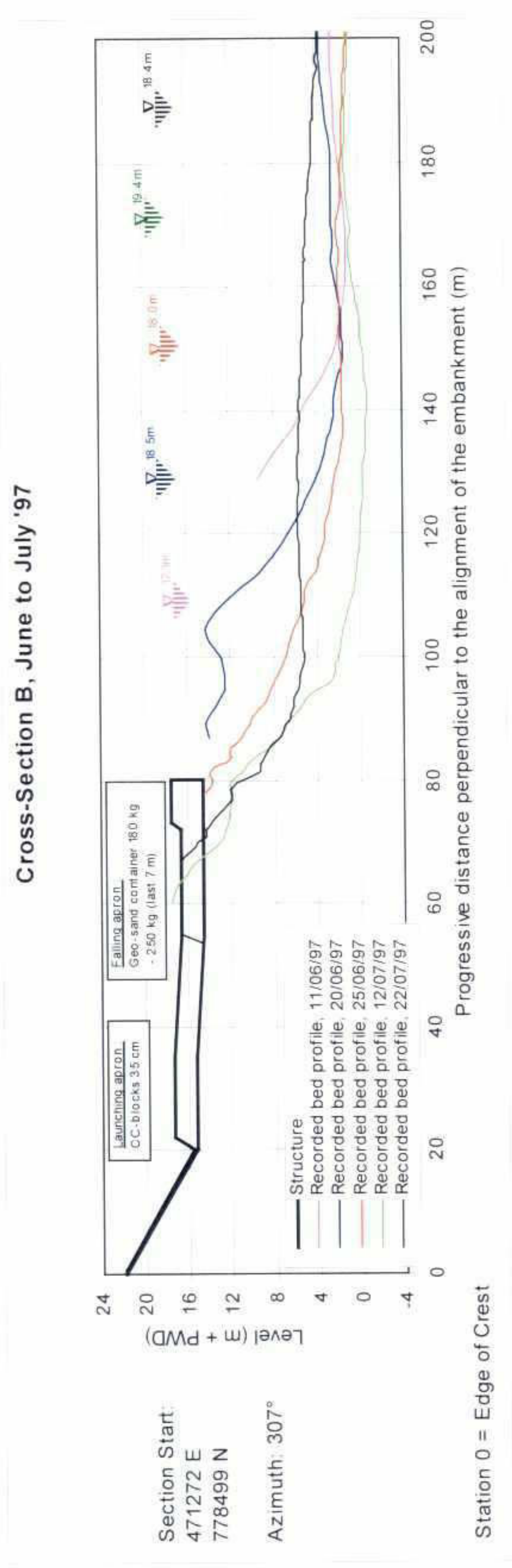


Fig. 4.2-22: Bathymetry and flow in July 1999

4.3 CROSS SECTIONS

Selected cross section surveys are presented in the following figures. Fig. 4.3-1 to Fig. 4.3-6 show the channel development in front of the test structure at the centre of Section B to Section G in 1997. The corresponding cross sections of the following years 1998 and 1999 are presented in Fig. 4.3-7 to Fig. 4.3-15. From 1998 Section E was split in Section E-1 and E-2 because of the different material of the falling apron used at that section. Since the main attack was concentrated on the downstream part of the structure in these years, Section H-1 and H-2 were surveyed as well.

The steepest slope inclination was measured from Section B to Section G on July 12, 1997. In Section H-1 and H-2 the steepest slope was surveyed on May 27, 1998. The parts from the falling apron to the riverbed of these sections are presented separately; thereby the horizontal and the vertical scale are chosen the same as 1: 500 (Fig. 4.3-16 to Fig. 4.3-19).



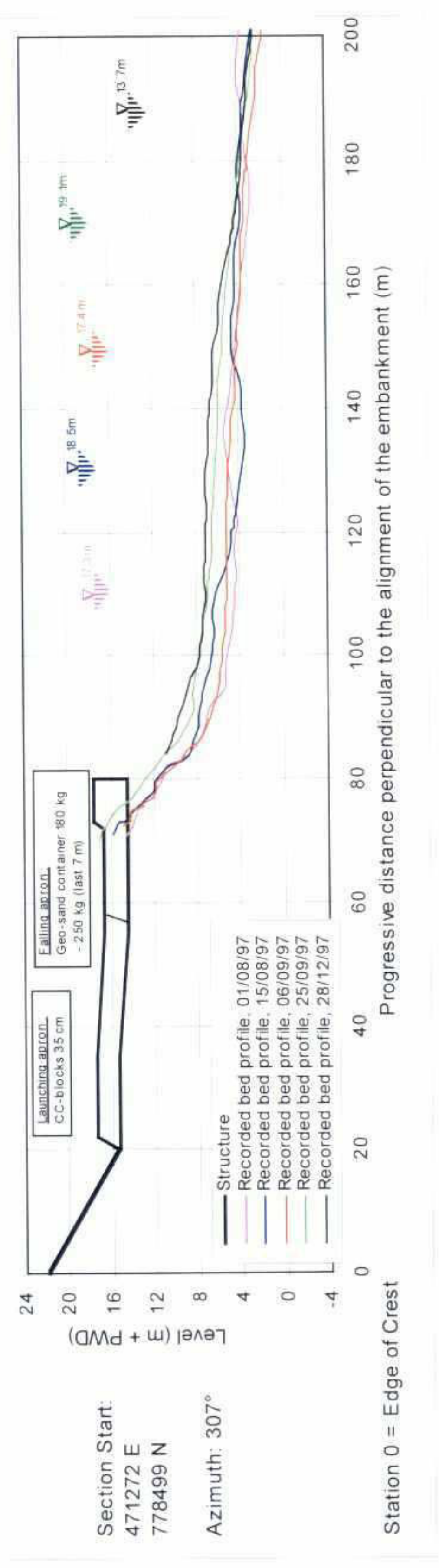


Fig: 4.3-1: Bathymetry cross section B in 1997

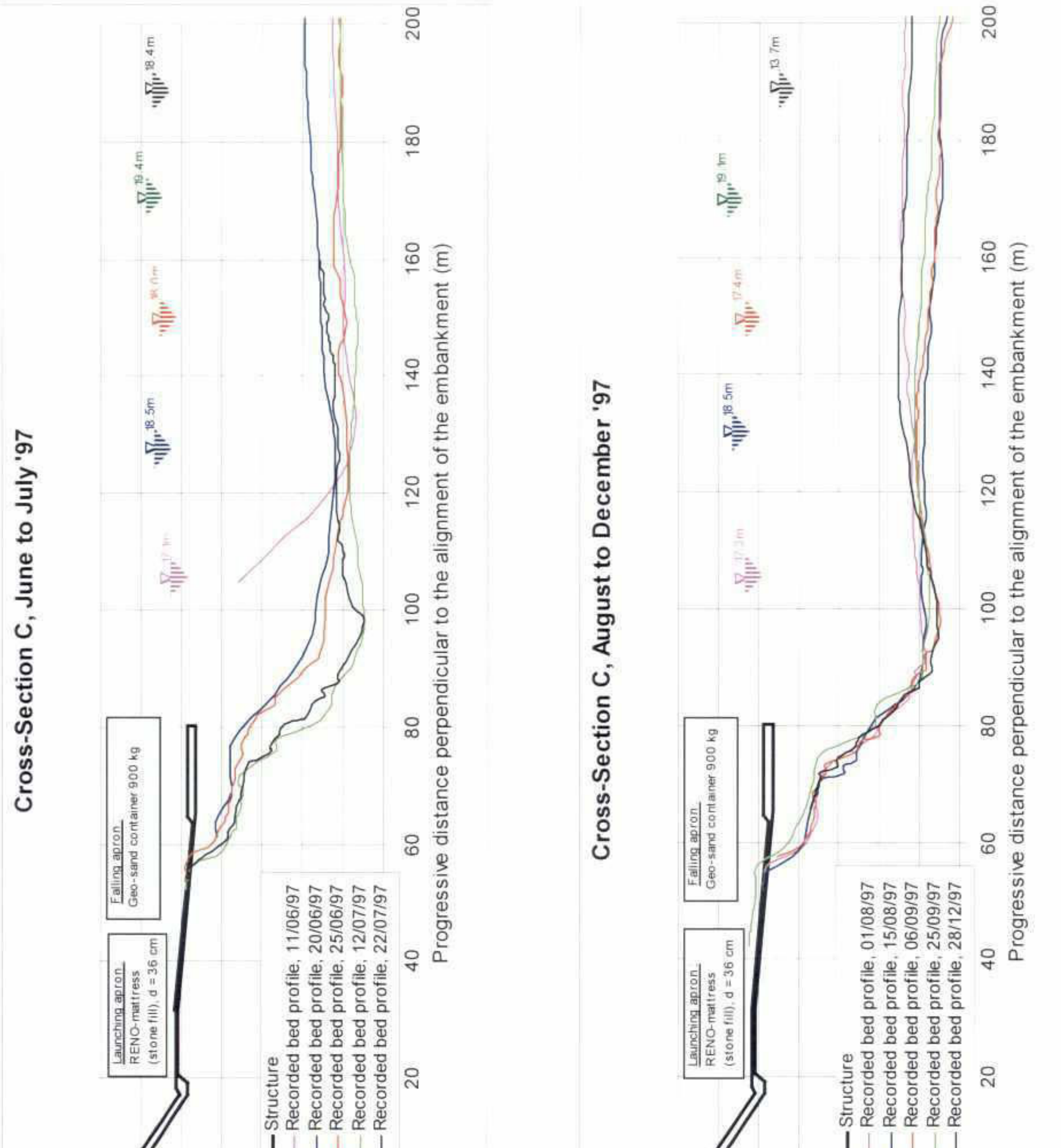


Fig: 4.3-2: Bathymetry cross section C in 1997

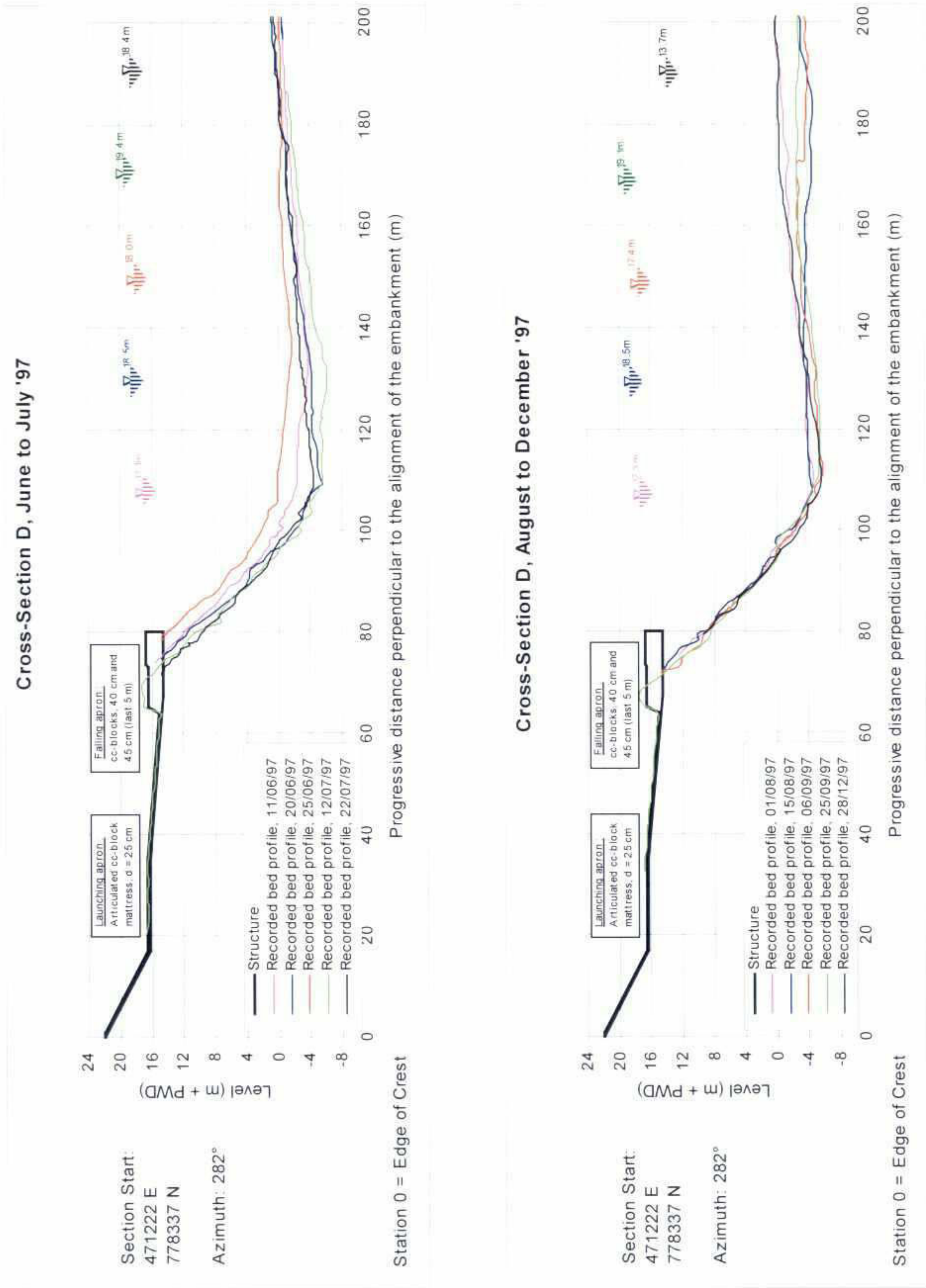


Fig: 4.3-3: Bathymetry cross section D in 1997

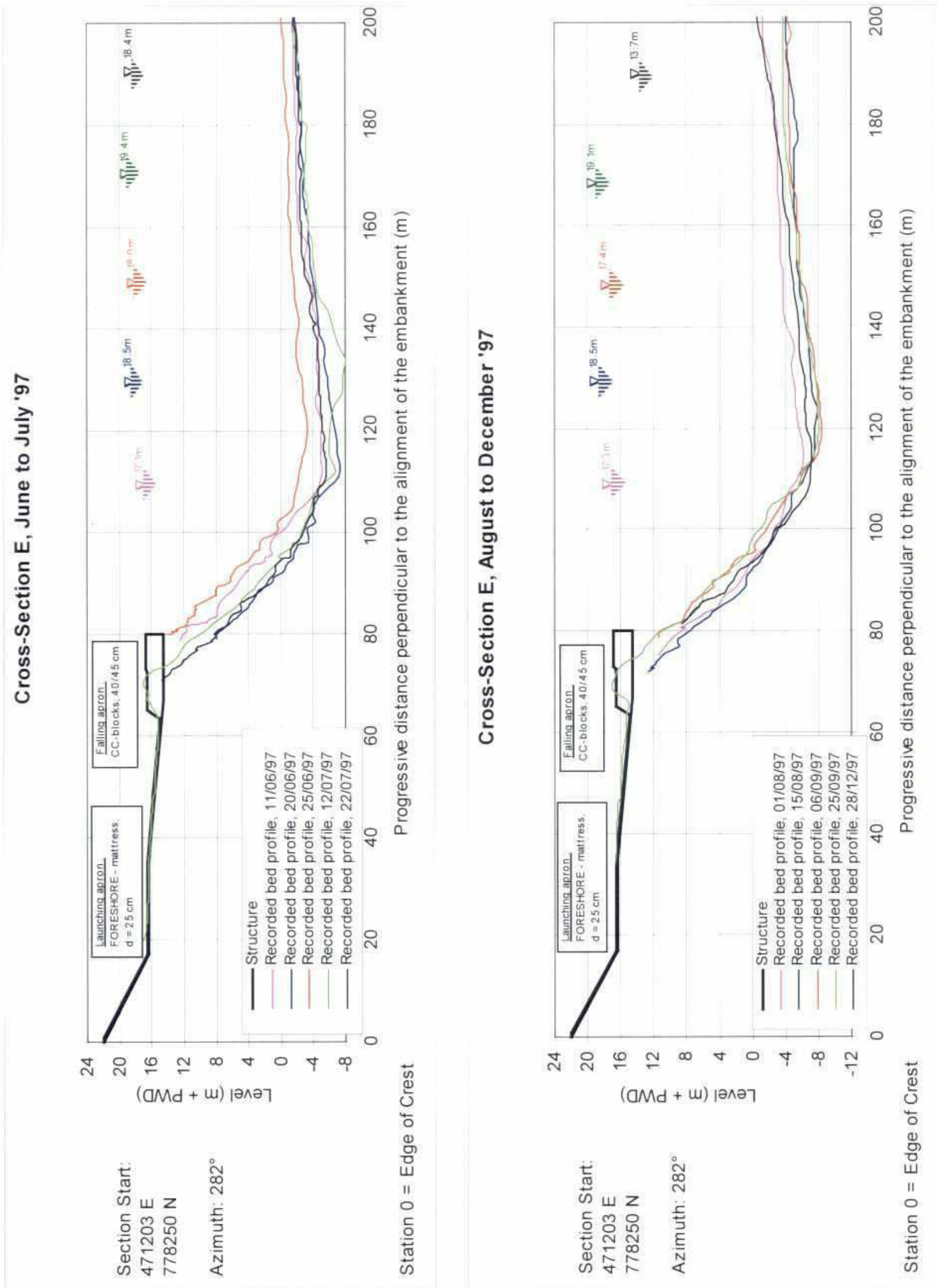


Fig: 4.3-4: Bathymetry cross section E in 1997

86

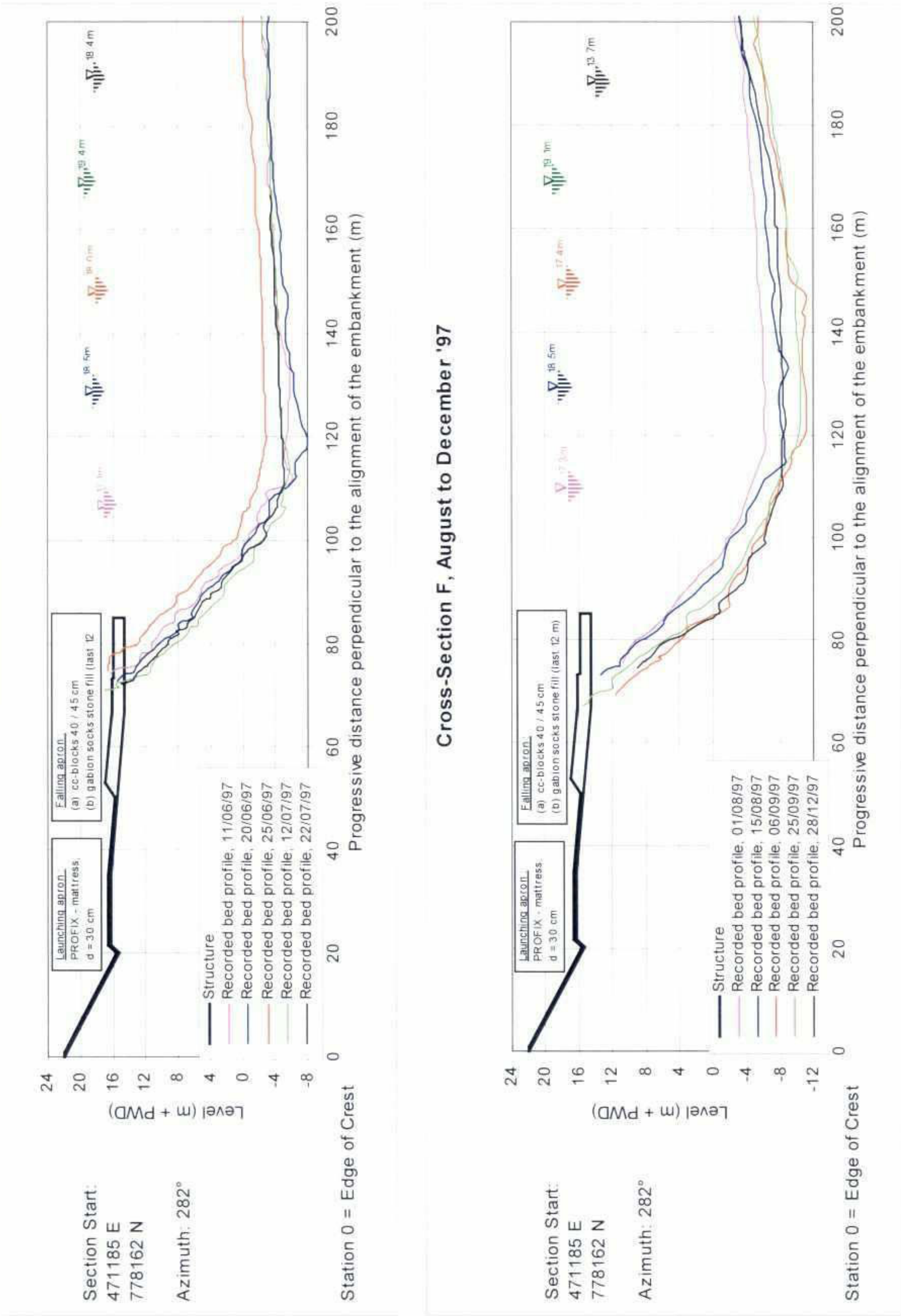


Fig: 4.3-5: Bathymetry cross section F in 1997

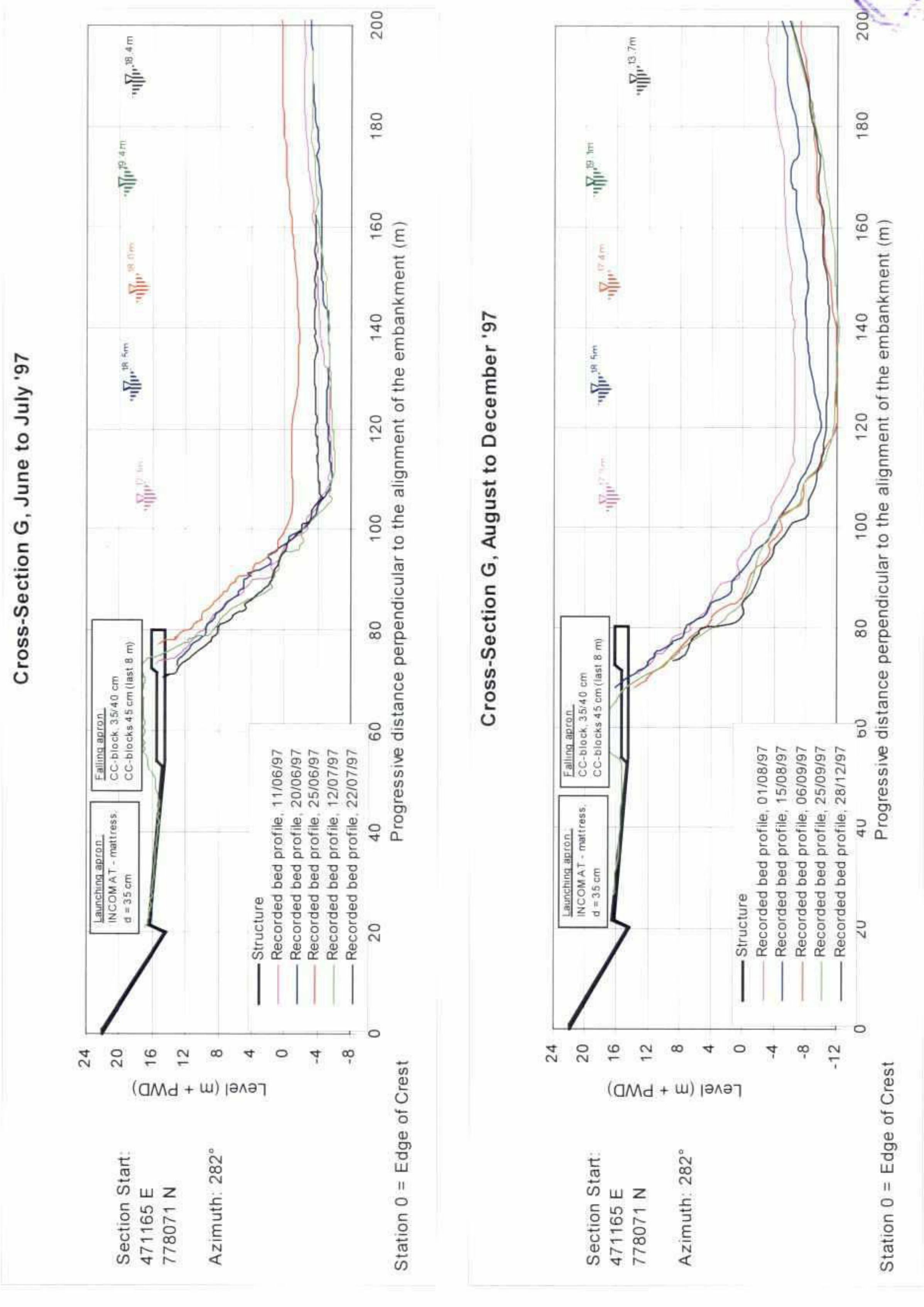
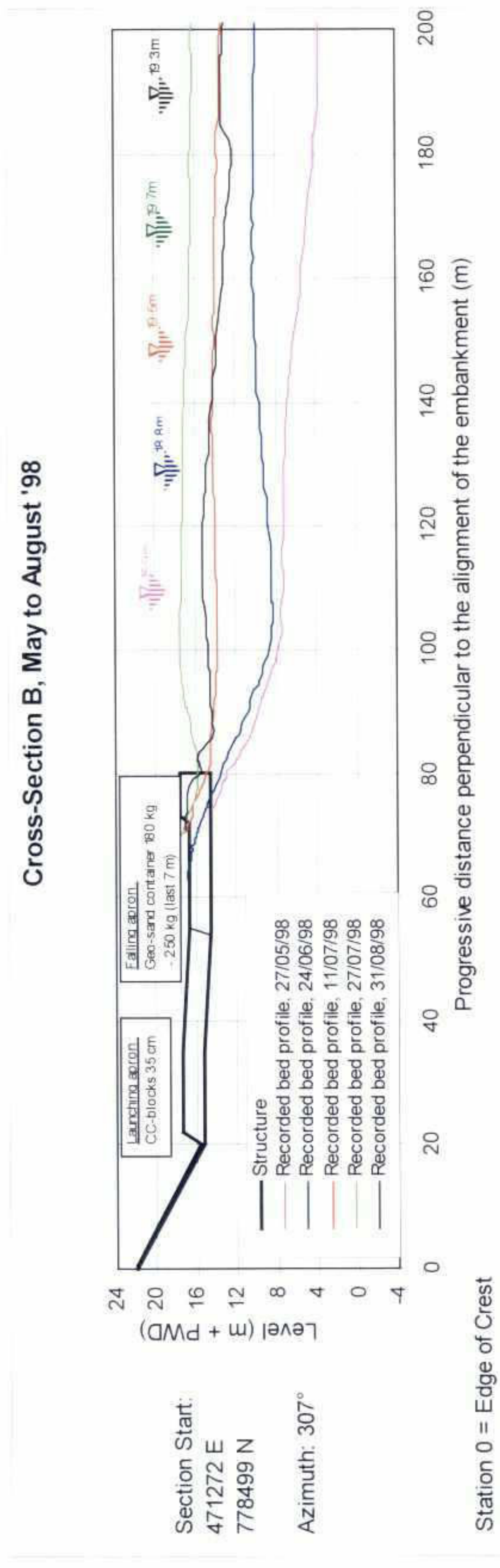


Fig: 4.3-6: Bathymetry cross section G in 1997



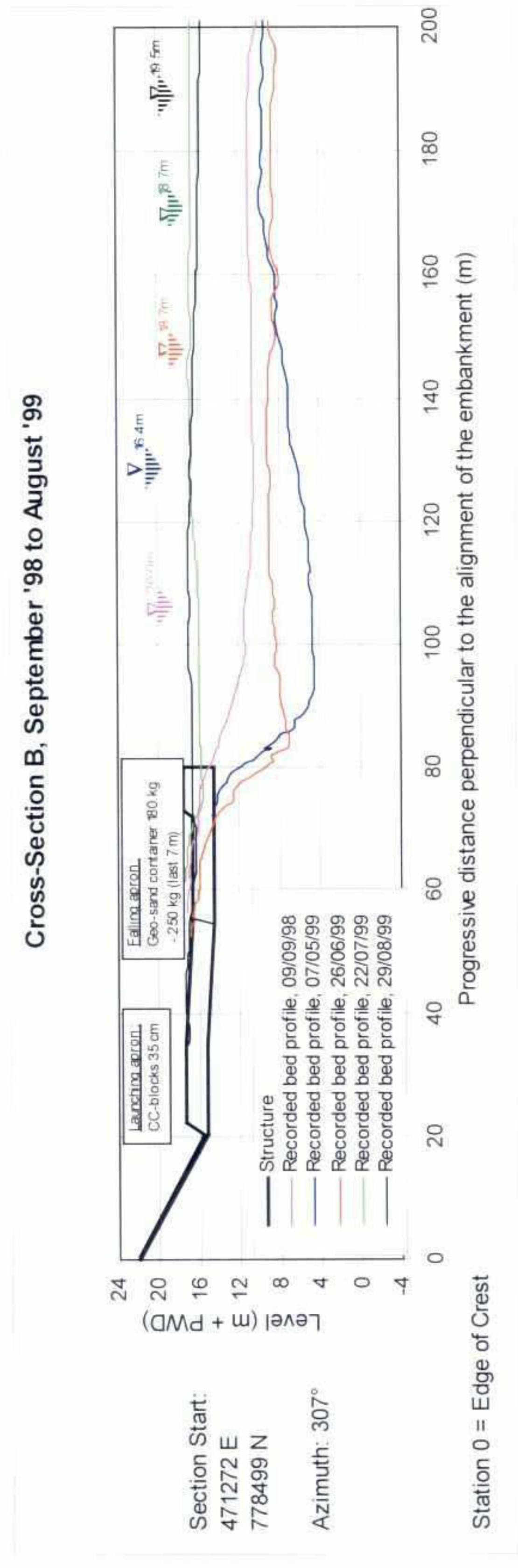
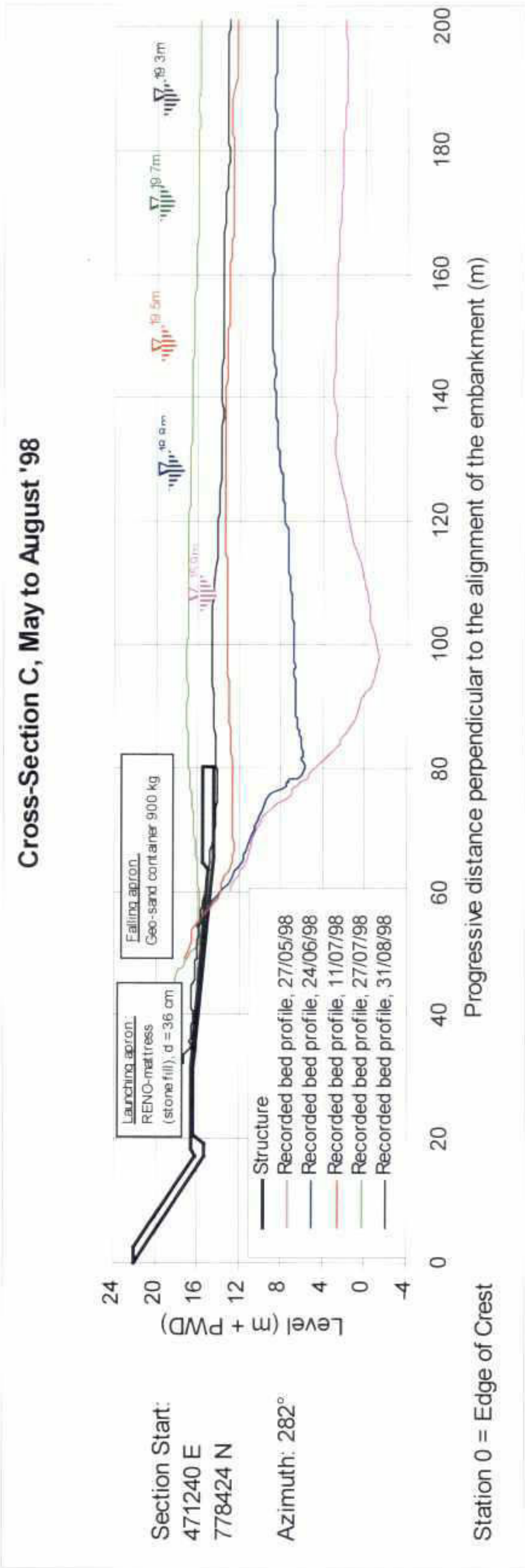


Fig: 4.3-7: Bathymetry cross section B in 1998, 1999



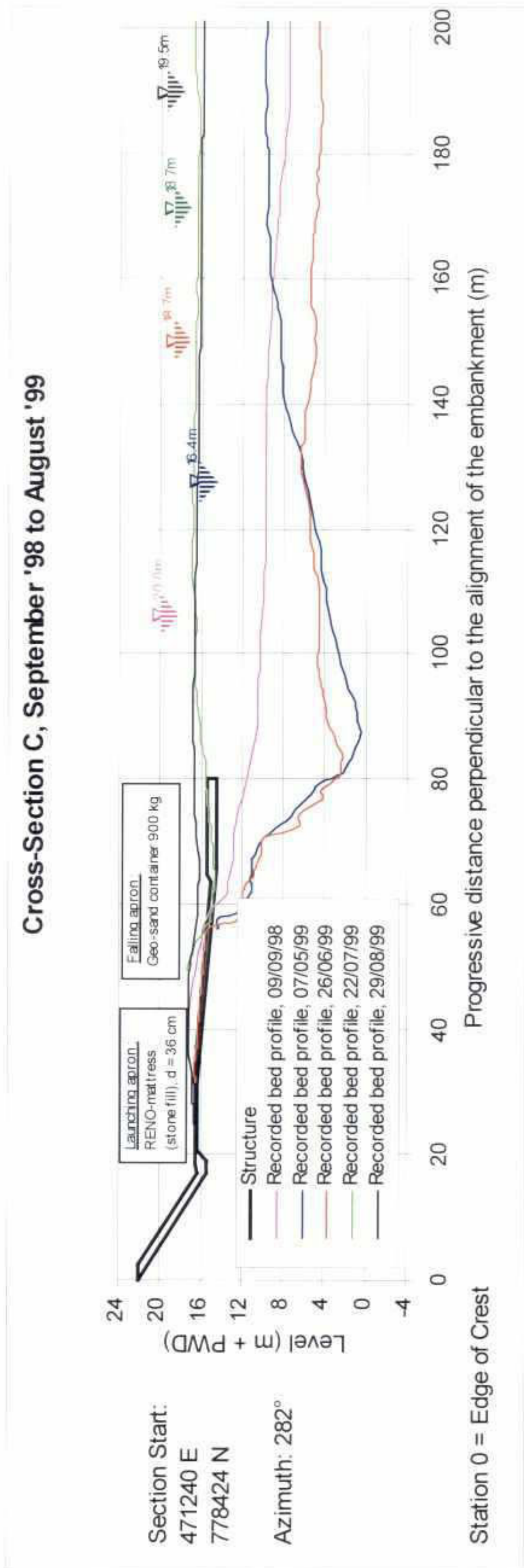


Fig: 4.3-8: Bathymetry cross section C in 1998, 1999

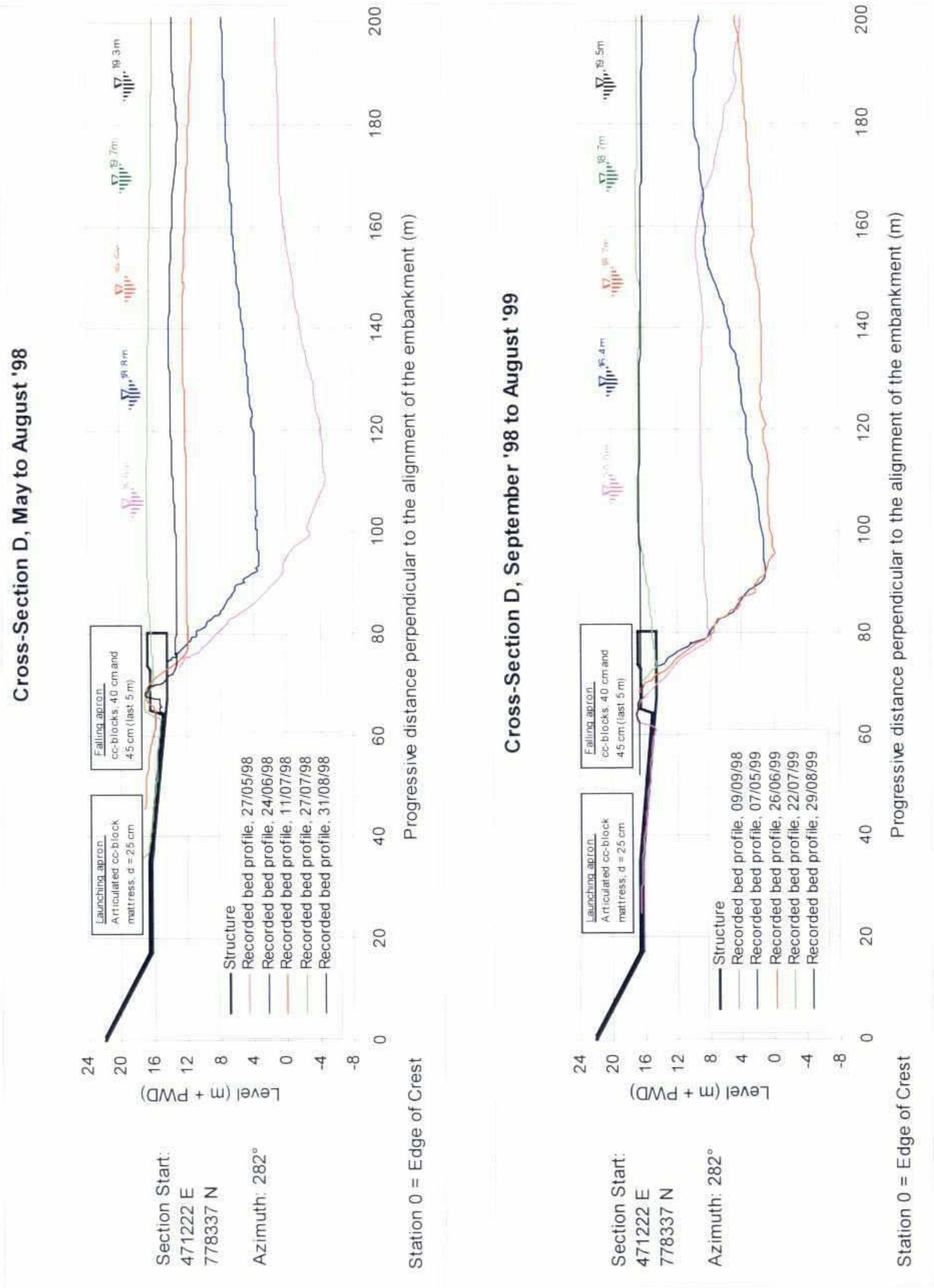


Fig: 4.3-9: Bathymetry cross section D in 1998, 1999

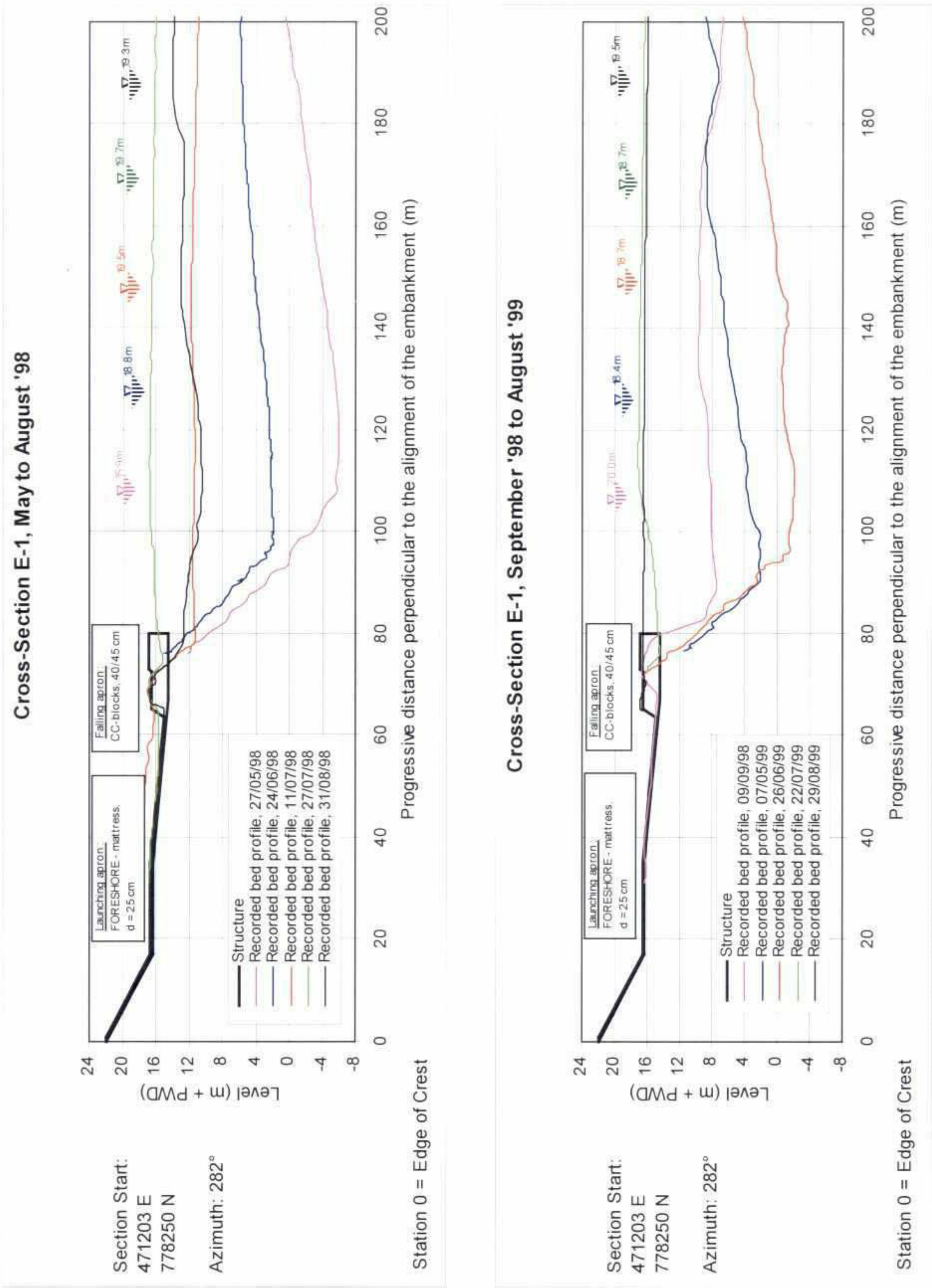
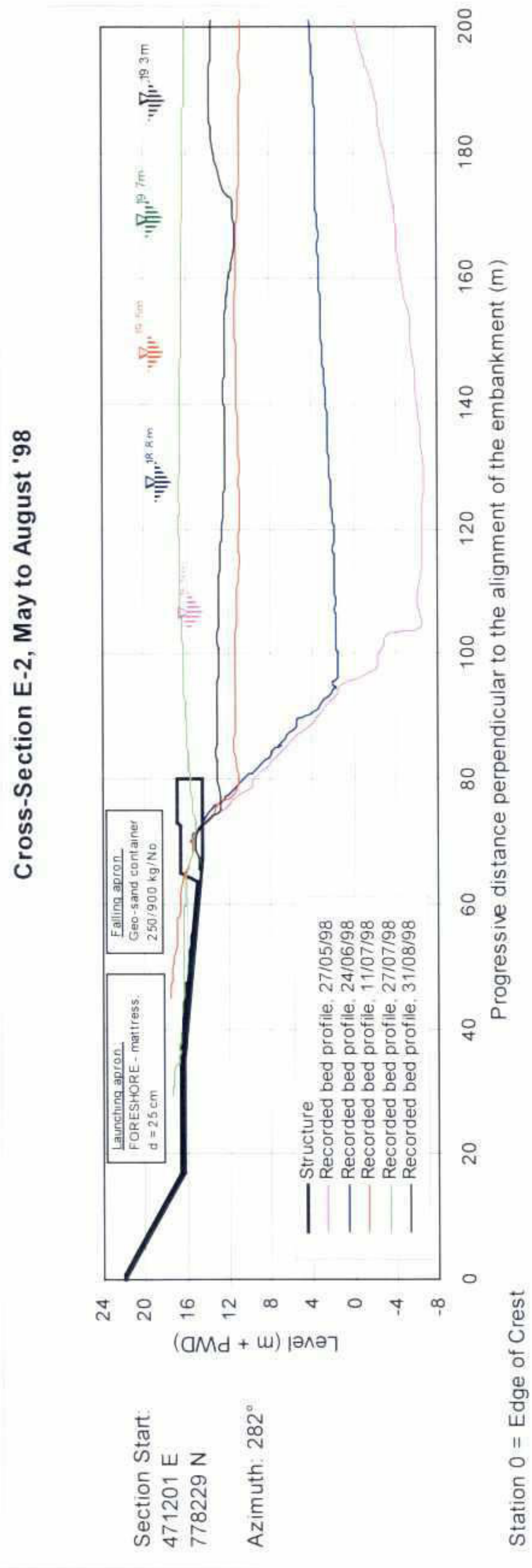


Fig: 4.3-10: Bathymetry cross section E1 in 1998, 1999



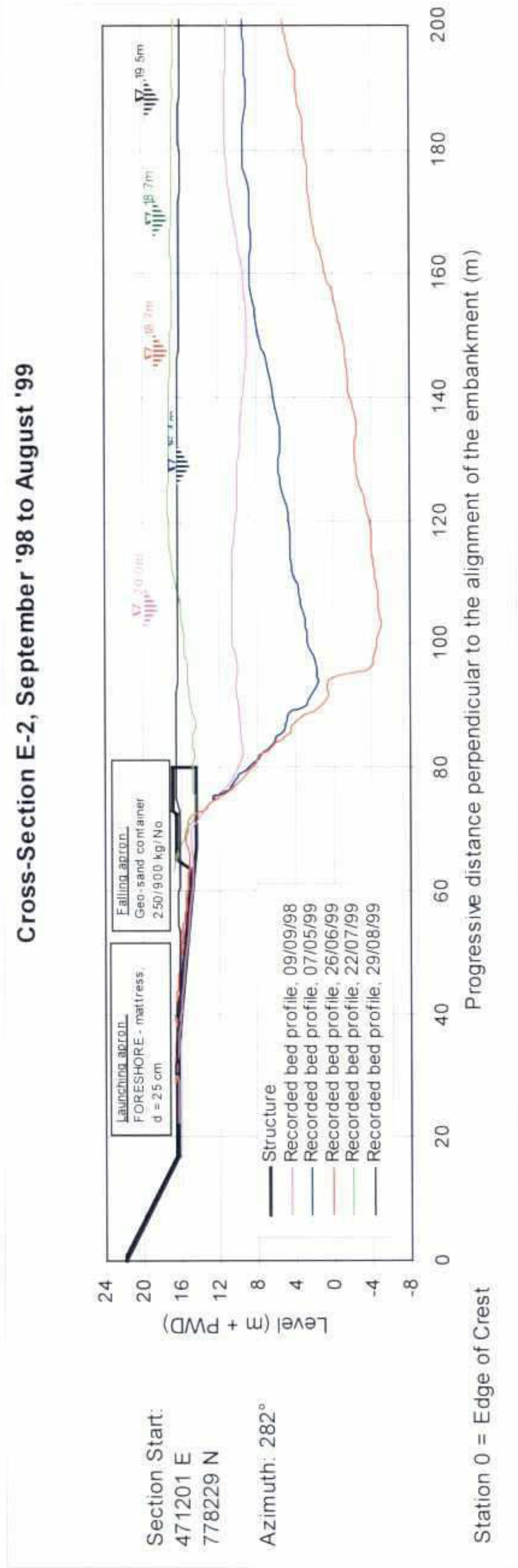
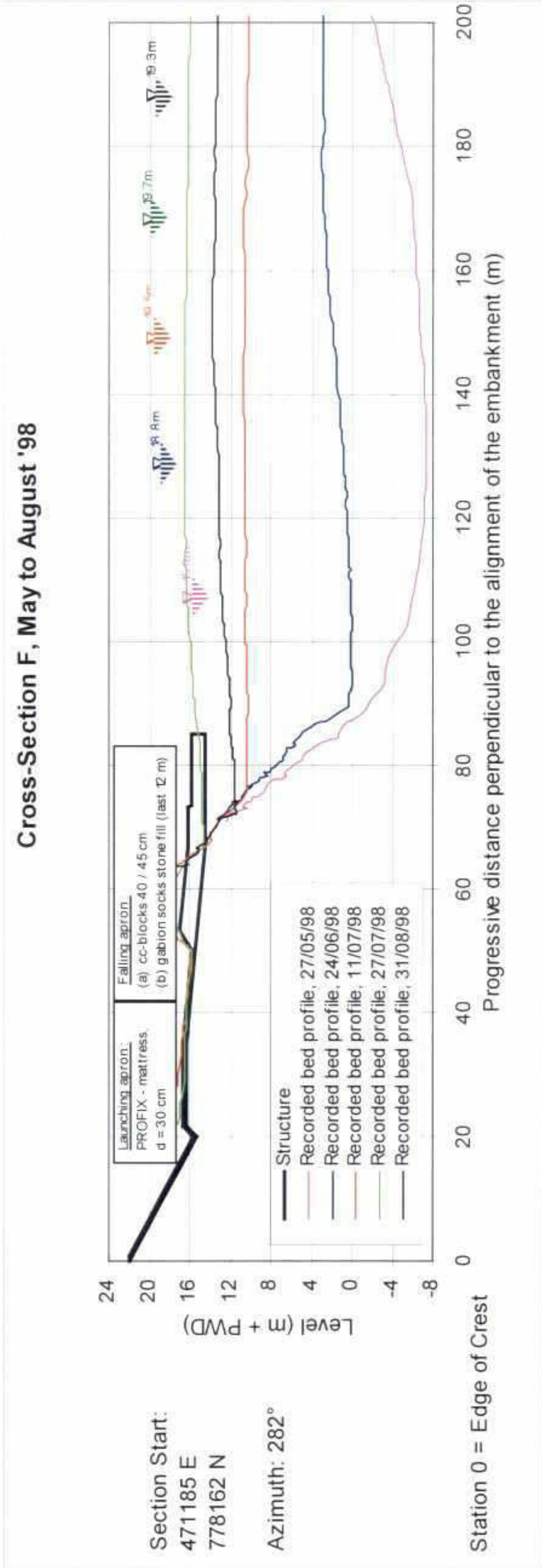


Fig: 4.3-11: Bathymetry cross section E2 in 1998, 1999



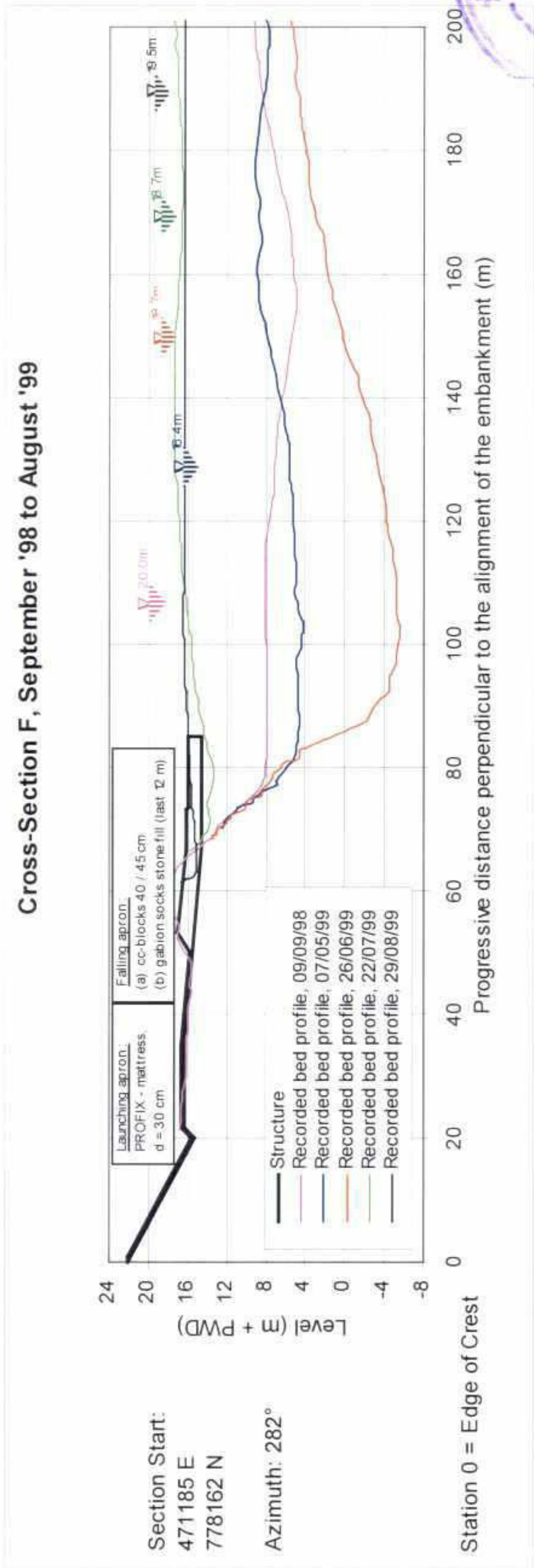


Fig: 4.3-12: Bathymetry cross section F in 1998, 1999

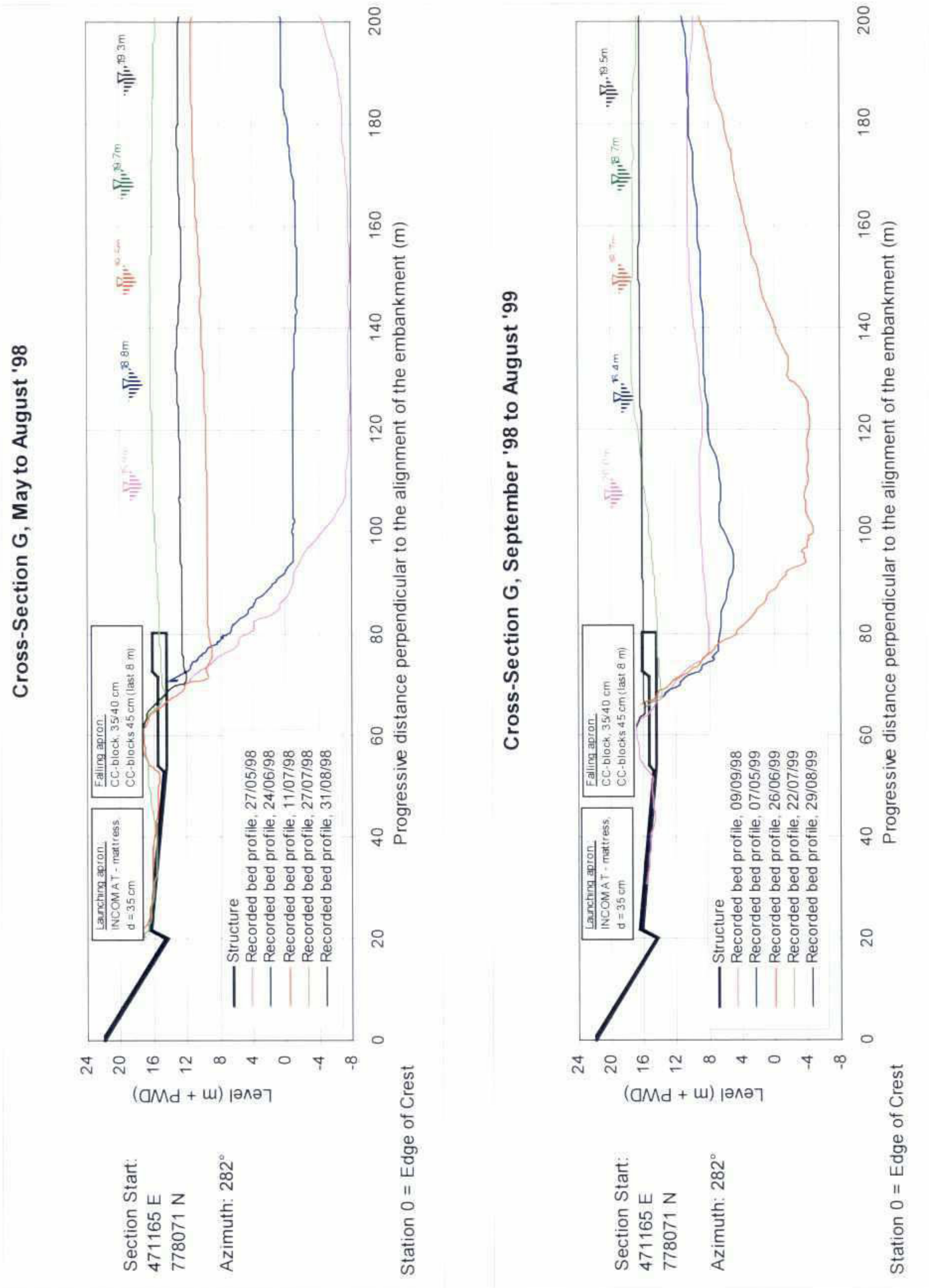


Fig: 4.3-13: Bathymetry cross section G in 1998, 1999

772

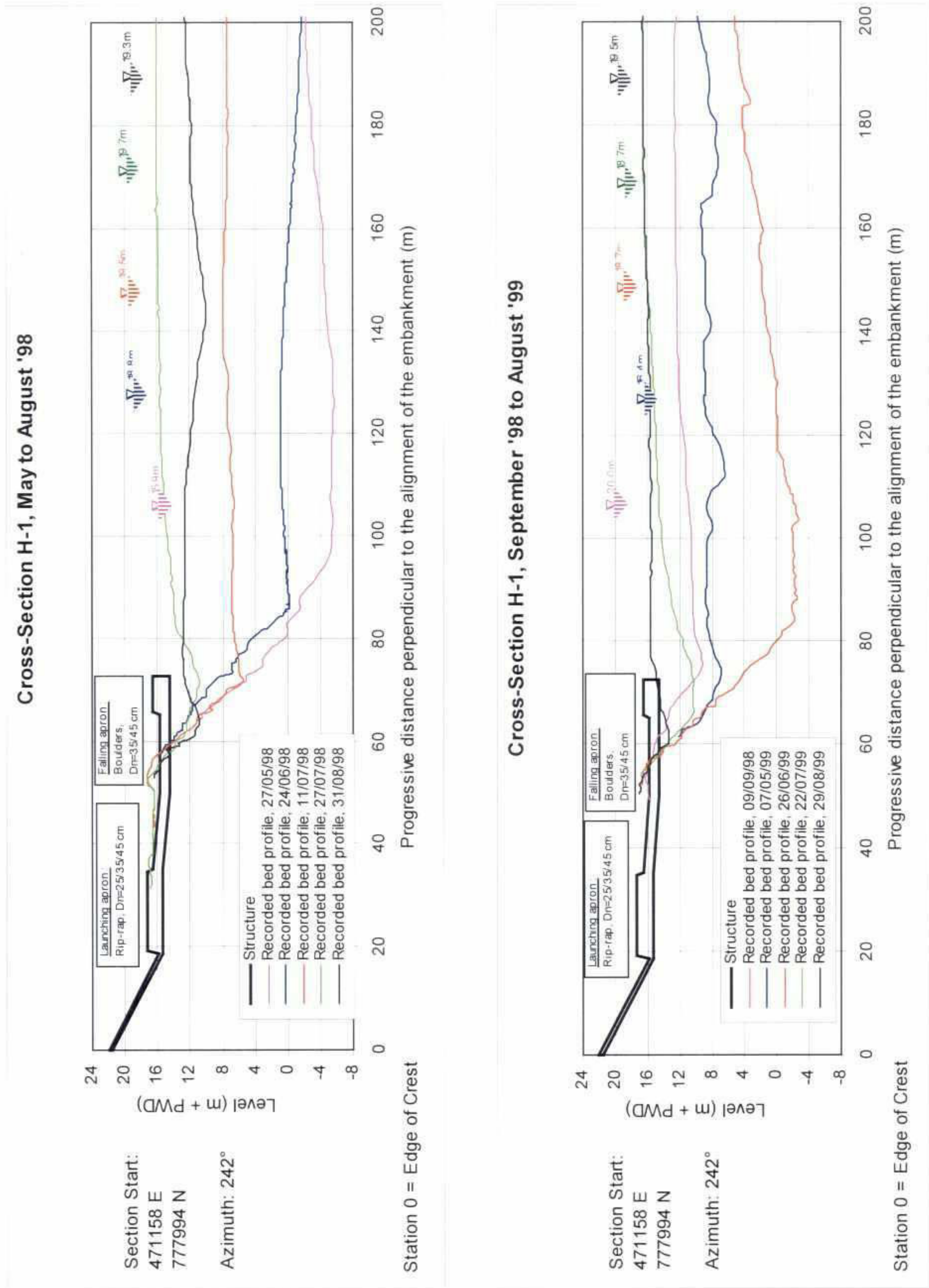
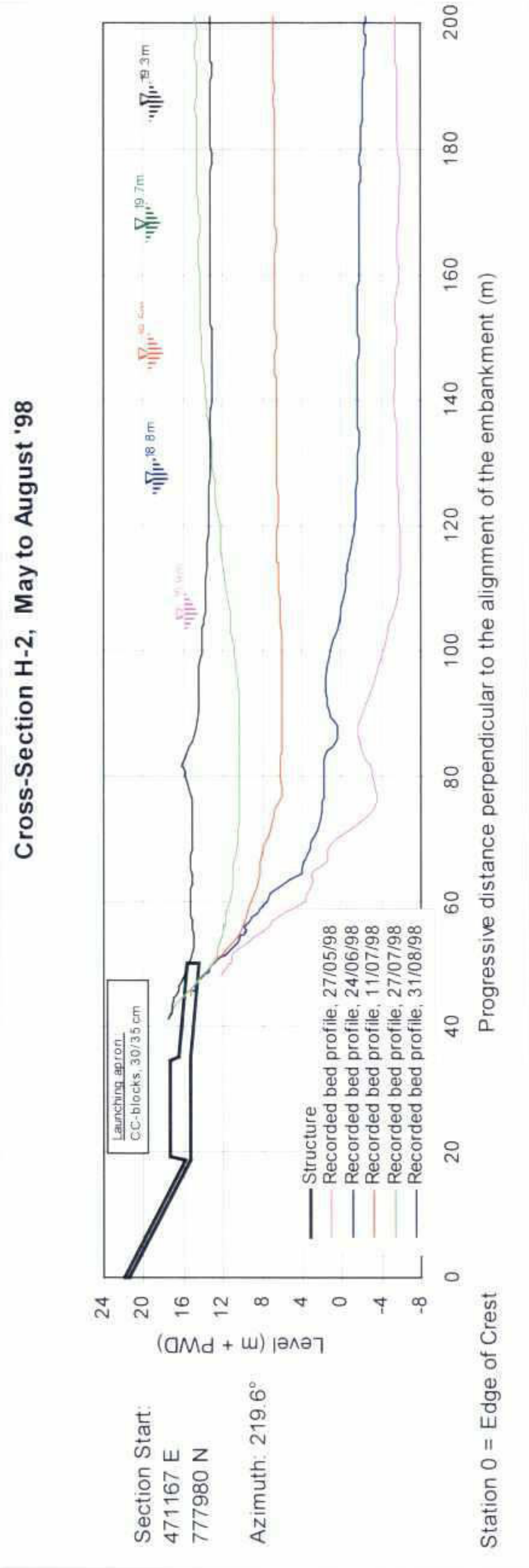


Fig: 4.3-14: Bathymetry cross section H1 in 1998, 1999



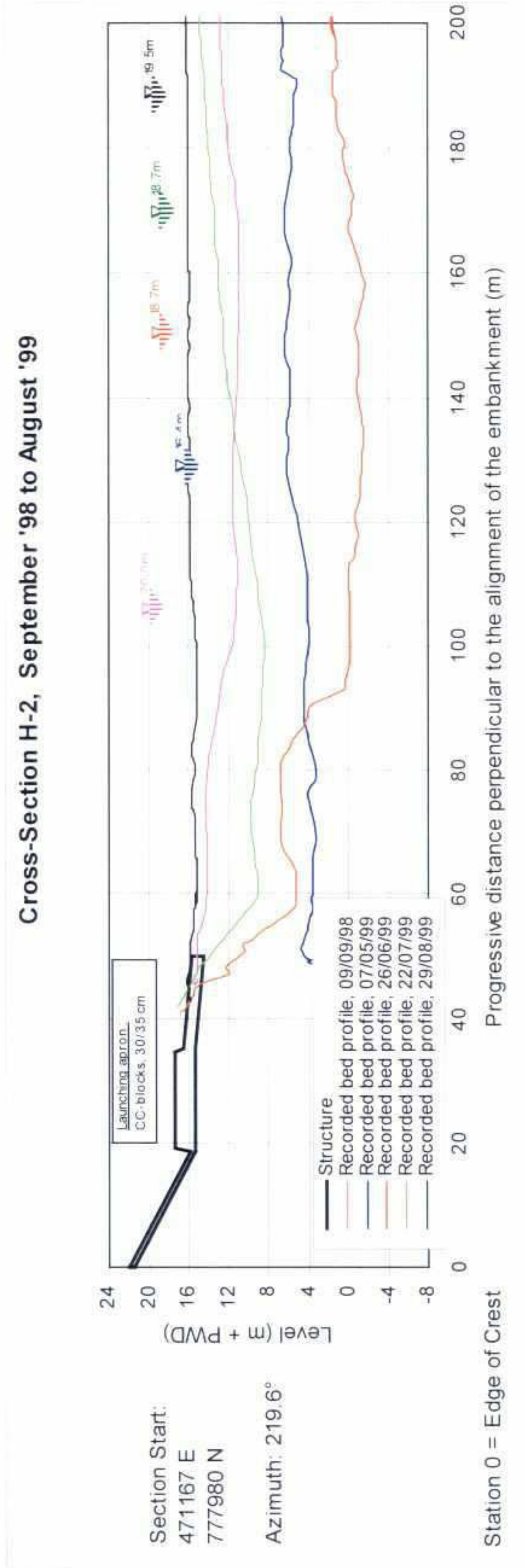
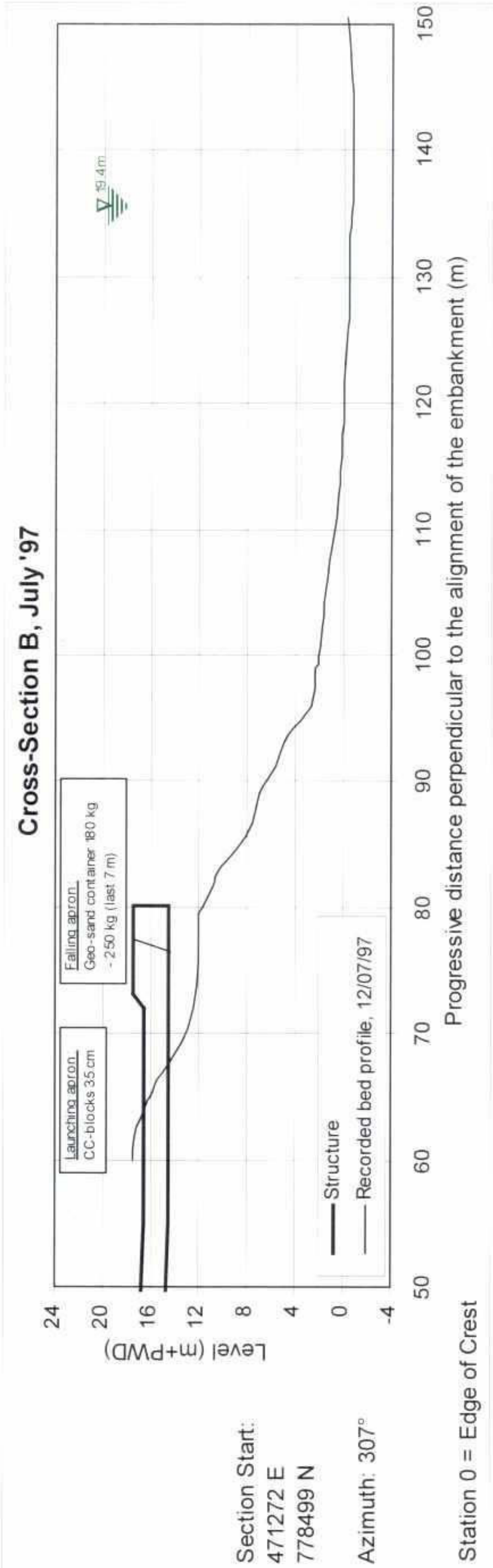


Fig: 4.3-15: Bathymetry cross section H2 in 1998, 1999



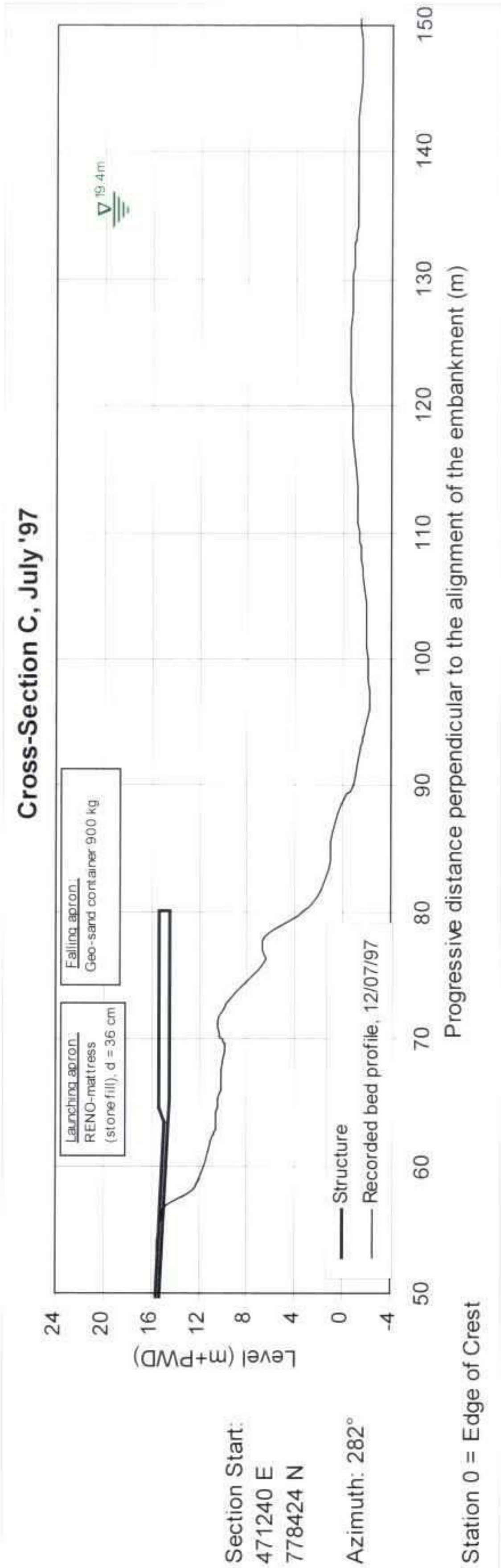


Fig: 4.3-16: Steepest slope surveyed in Section B and C

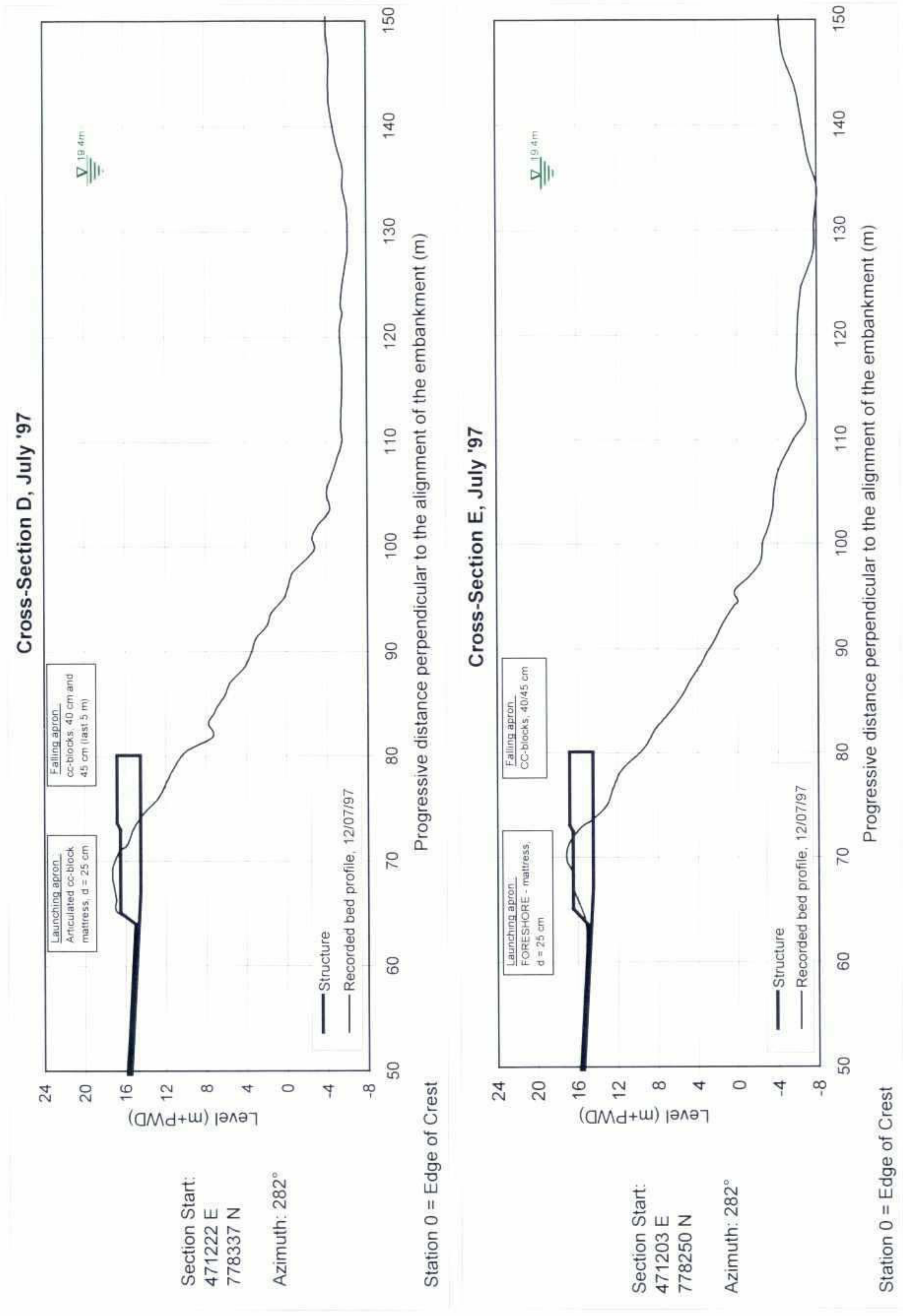


Fig: 4.3-17: Steepest slope surveyed in Section D and E

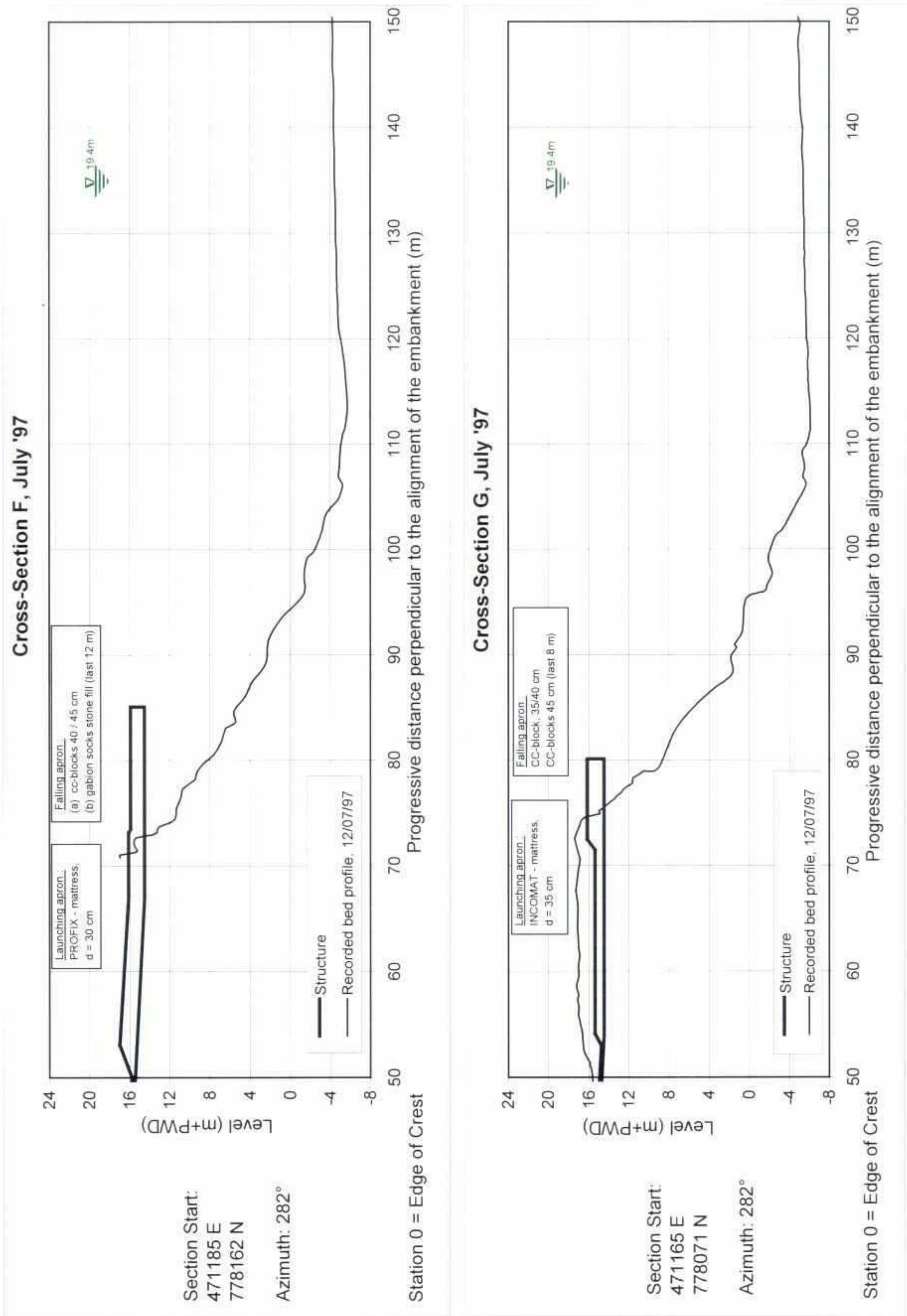
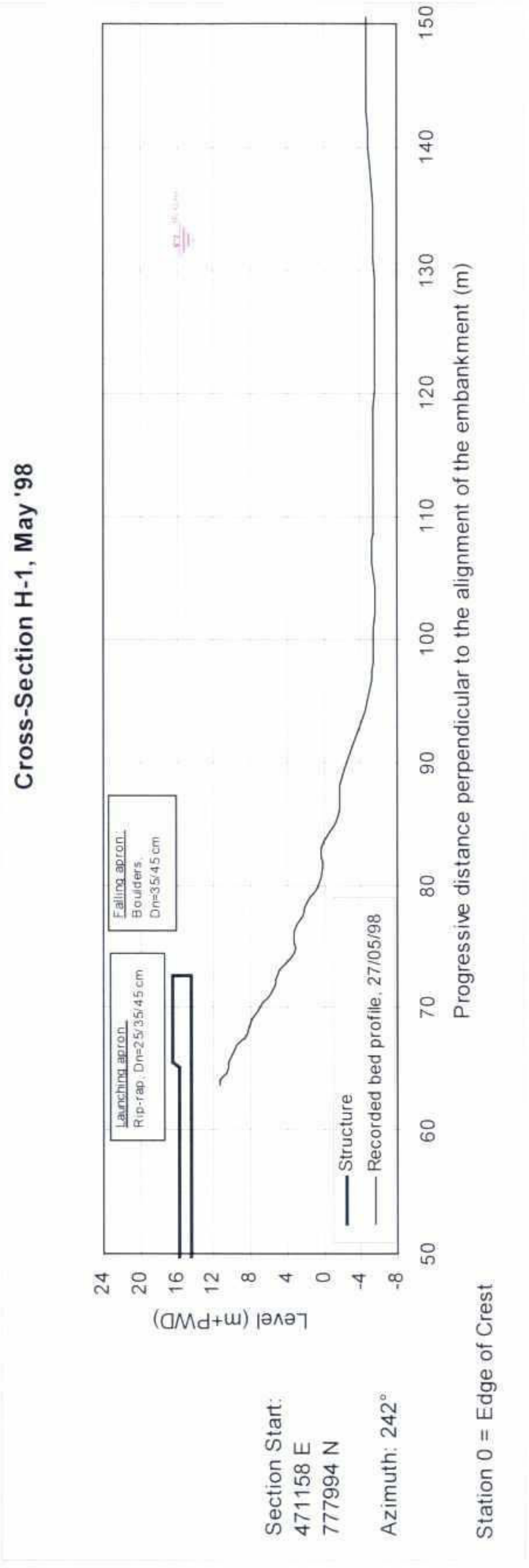


Fig: 4.3-18: Steepest slope surveyed in Section F and G



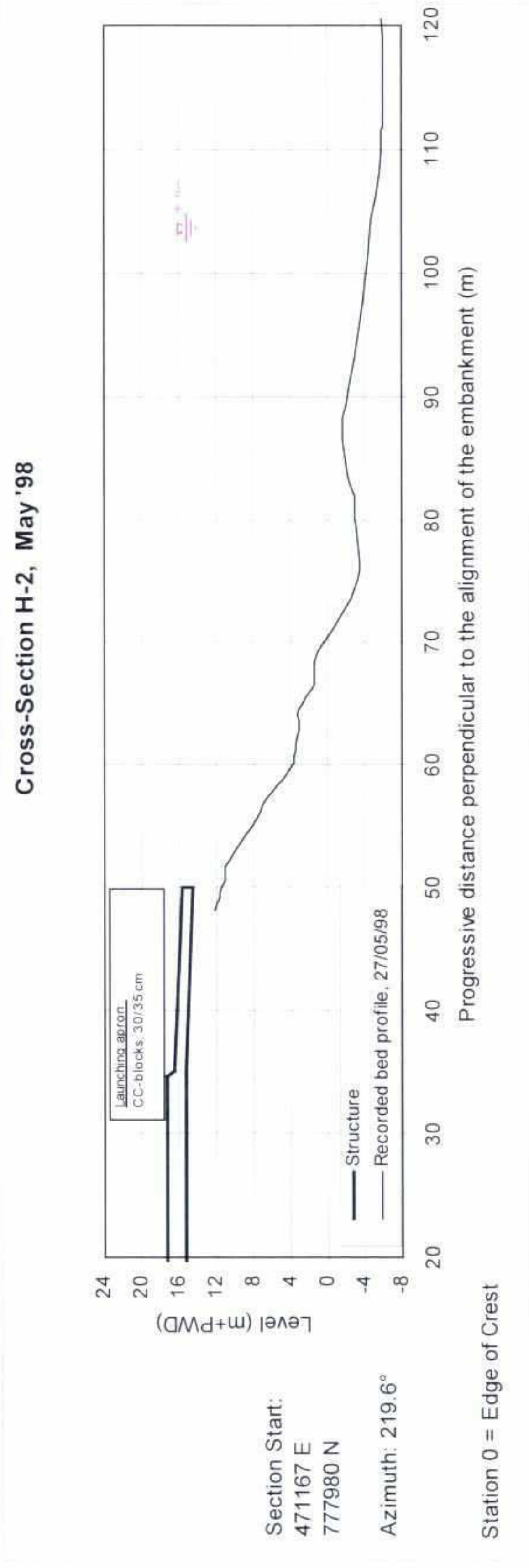


Fig: 4.3-19: Steepest slope surveyed in Section H-1 and H-2

4.4 DETAILED FLOW INVESTIGATIONS

4.4.1 Advanced DGPS Float Tracking

Three surveys are selected to show the flow pattern in front of the structure. The first shown survey (Fig. 4.4-1 to Fig. 4.4-3) carried out on June 19, 1997 presents the flow at different depths in the main channel (surface, 3 m and 6 m). The water level reached that time its first peak at 18.6 m+PWD in the monsoon 1997.

The flow velocities show no significant difference regarding the depth at which they were measured. But a significant reduction by the depth could be found in the comparison of the western flow line in the more shallow area (5 to 10 m+PWD). Here the flow was more than 3 m/s at the surface and in 6 m depth the flow is reduced to about 2.5 m/s.

The surface flow lines just in front of the structure show that they are diverging at the downstream termination but the corresponding flow lines at 6 m depth are converging at that place. The highest velocities were found 200 m off from the end of the falling apron along the centre of the main channel. The tracks along the falling aprons indicate much lower velocities than in the main channel.

The second selected survey of August 12, 1997 was carried out at the rising water level stage three days before the water level peak (Fig. 4.4-4). The flow was measured at a depth of 3 m.

Since the bed level in front of the structure was less than in June, the flow velocities were reduced in the main channel. The track along the falling apron shows on the other hand higher velocities than in June. The velocity accelerated from 2 m/s at Section D to 3 m/s at Section H. The turbulence of the flow downstream from Section H were measured as described in the previous sections.

The last selected flow tracking was carried out during the first peak flow of 1998 on June 14 (Fig. 4.4-5). Although a higher water level was recorded at that time in comparison to the other two presented surveys, the flow velocities were in general not exceeding 2 m/s. The flow along the falling apron changed its direction towards east after passing the structure in the same way as the channel direction changed compared with the previous year.

4.4.2 Current Point Measurements

The difficulty to anchor the survey boat in front of the structure has been already described in Section 3.2. However on June 07, 1998 measurements were taken at all sections but at low surface velocities of about 1 m/s. The result of this survey is presented in Fig. 4.4-6. The length of the vectors represents the flow velocity (2 cm = 1 m/s). A significant reduction of the flow velocity along the water column was measured only at 80 % of the water depth, for instance at 8 m depth in 10 m water depth. Between 0.2 and 0.6 of the water depth only small variations were recorded. This result verifies the results from the float tracking at different depth. A significant reduction of the flow velocity along the water column began at a depth of 60 % of the water depth.

From Section B to Section G the flow direction changed with the depth towards west whereas in front of Section H the flow direction changed towards east.

One day after heavy rainfall (140 mm on June 21) at a higher water level stage (19.0 m+PWD) three verticals were measured at Section E-1 and one at Section F respectively. At that time the survey boat could be moored to the buoys (see Section 3.2) and higher velocities were recorded.

The results are presented together with the bathymetric cross section in Fig. 4.4-7 and Fig. 4.4-8. The significant reduction of the flow velocity above the falling apron could be surveyed along Section E-1.

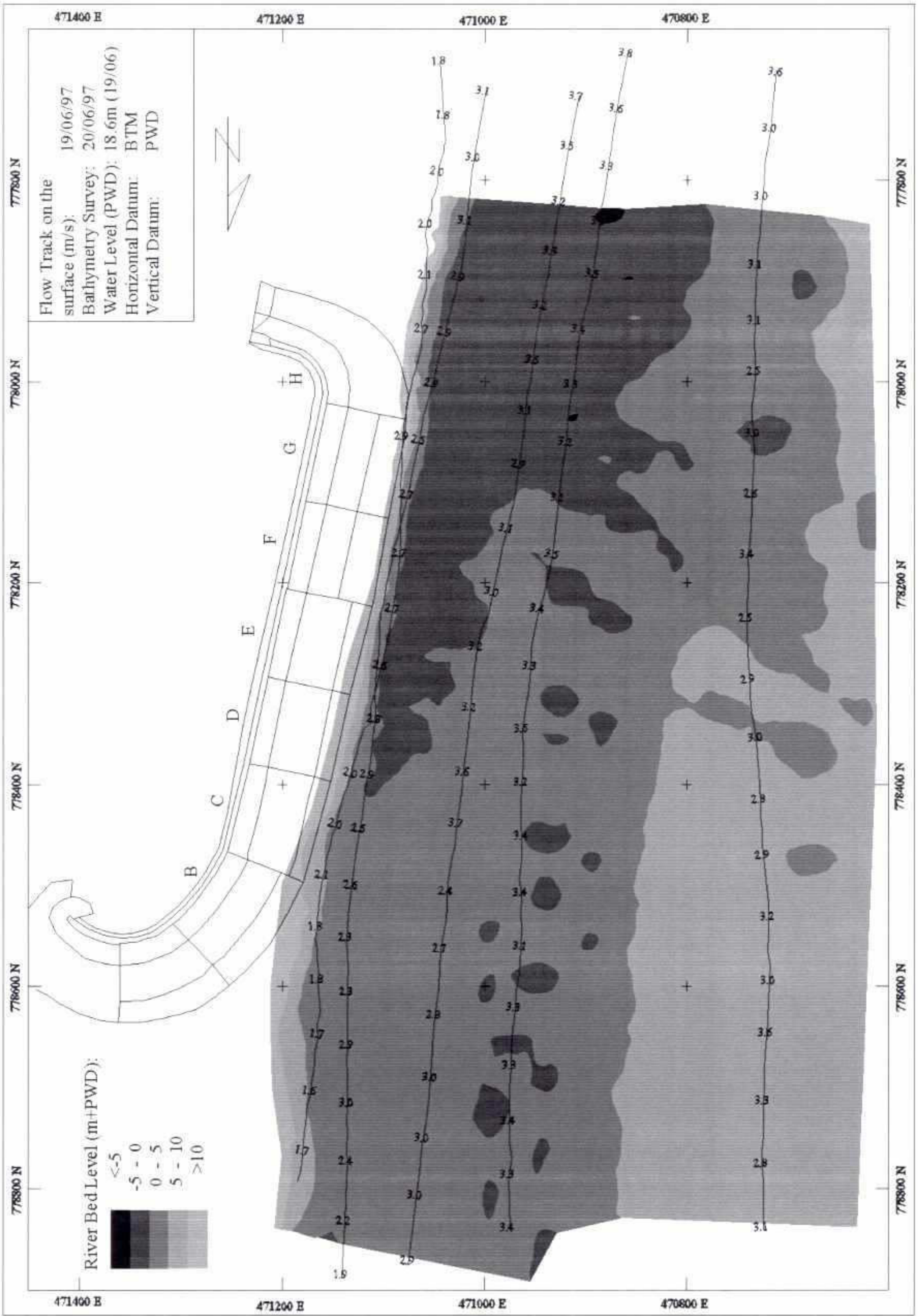


Fig. 4.4-1: Flow pattern at Test Site II in June 1997

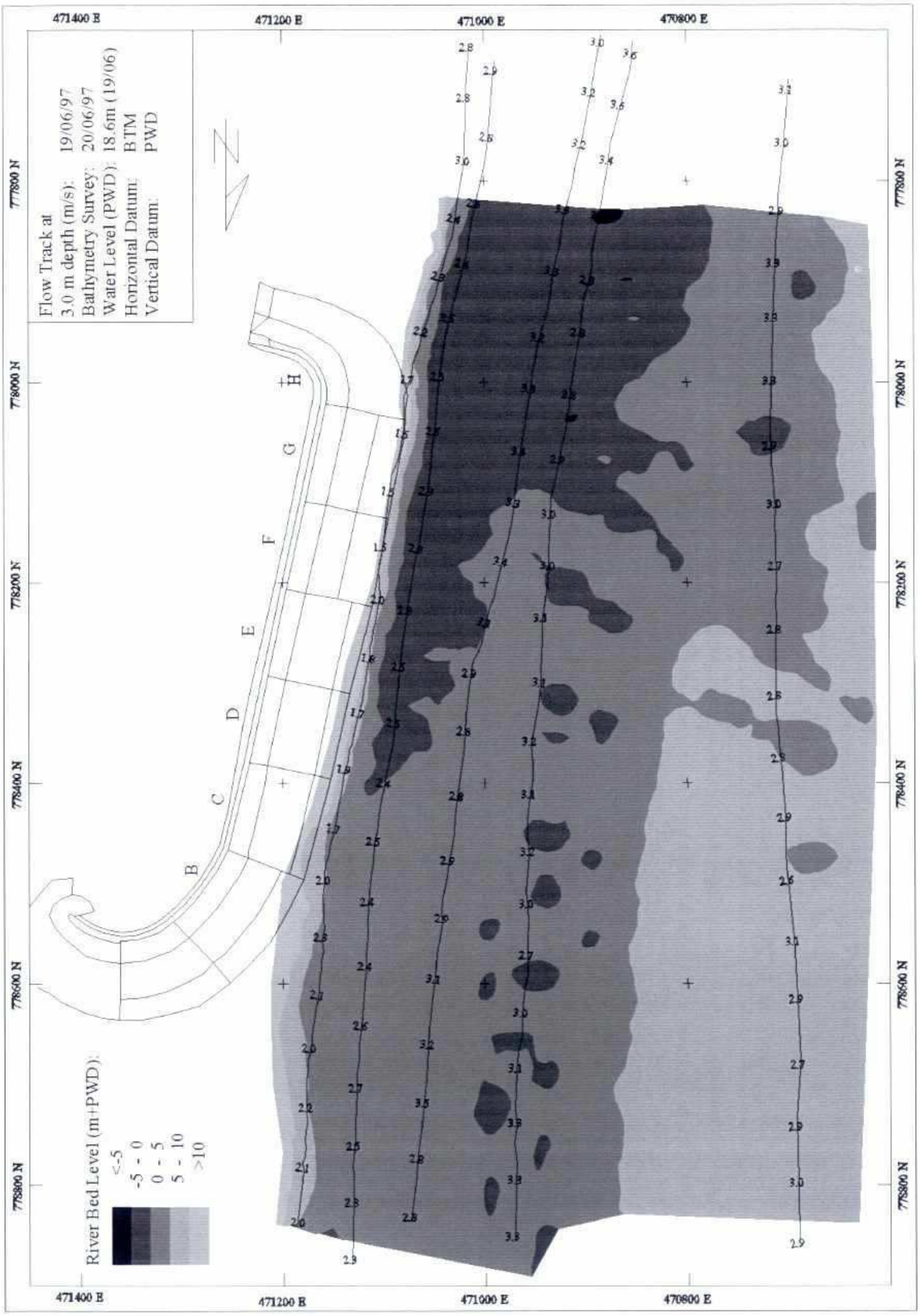


Fig. 4.4-2: Flow pattern at Test Site II in June 1997

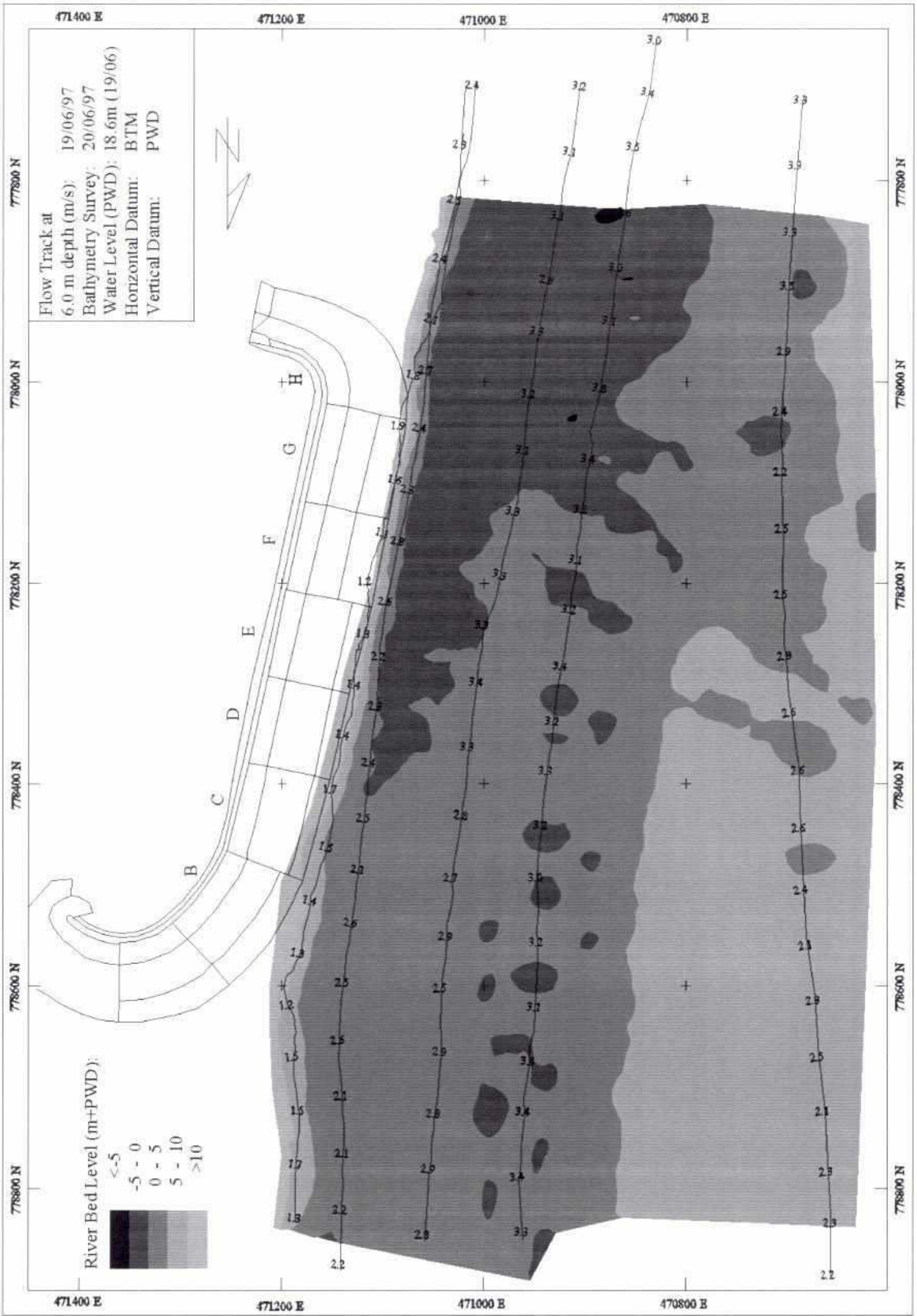


Fig. 4.4-3: Flow pattern at Test Site II in June 1997

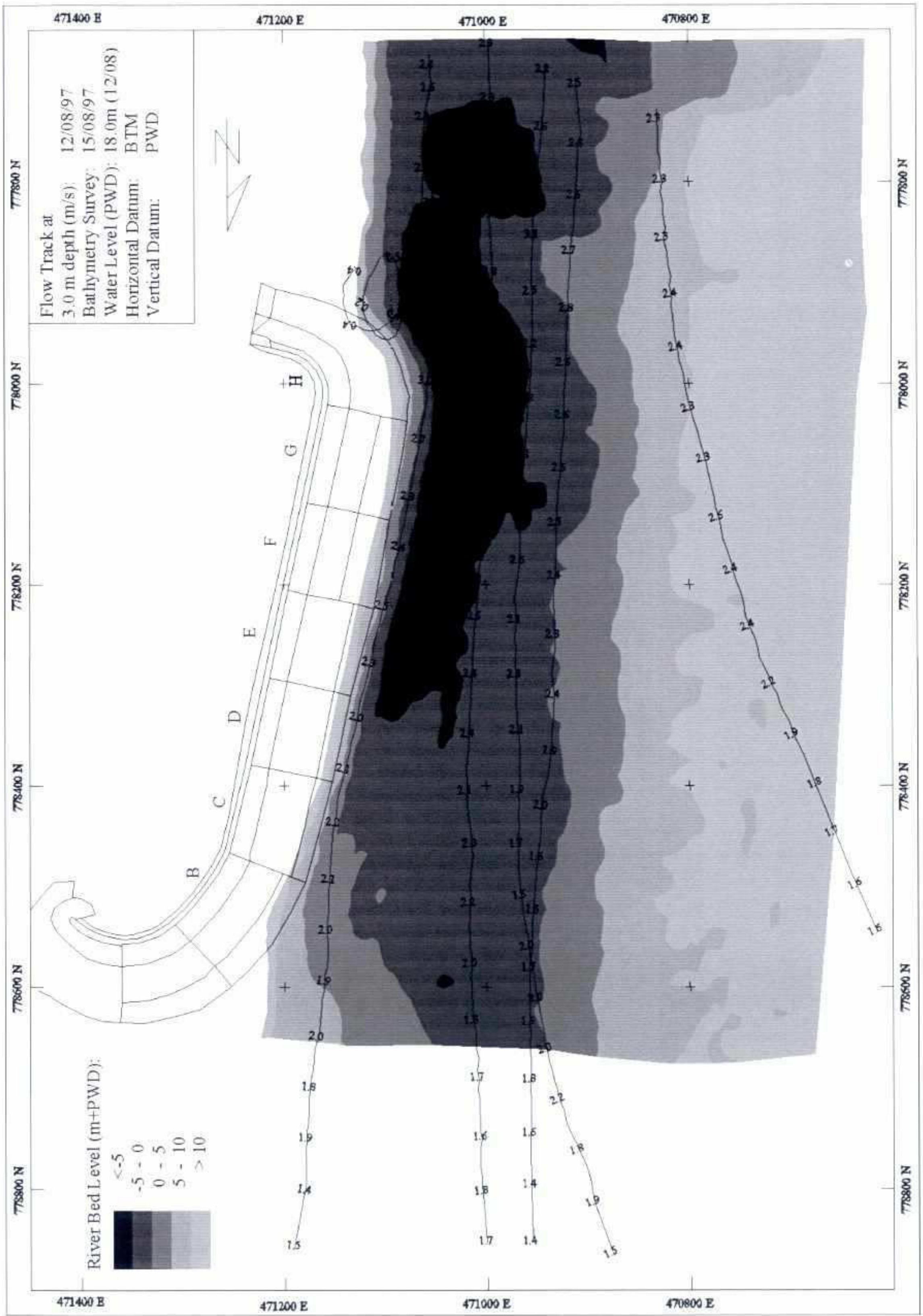


Fig. 4.4-4: Flow pattern at Test Site II in August 1997

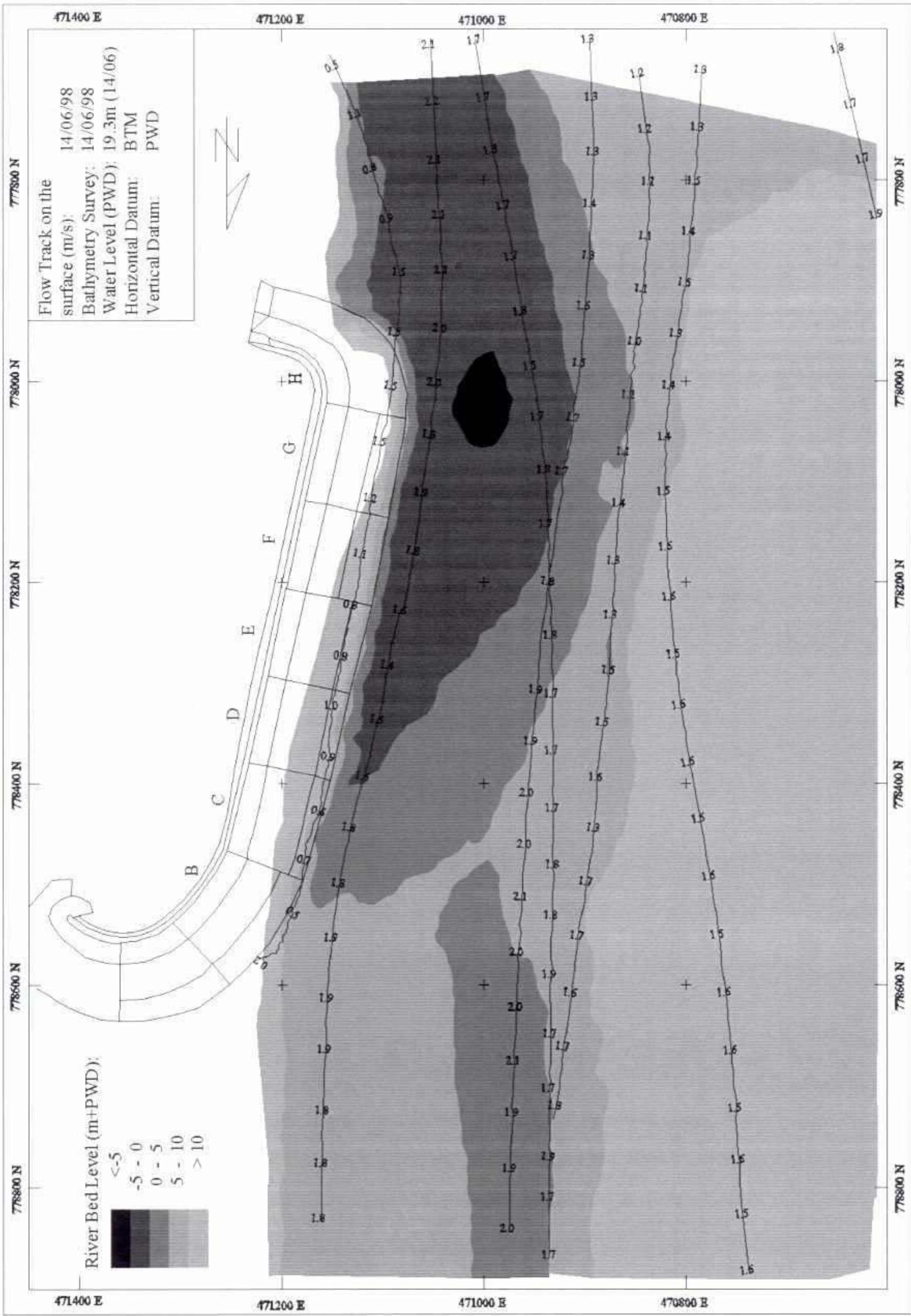


Fig. 4.4-5: Flow pattern at Test Site II in June 1998

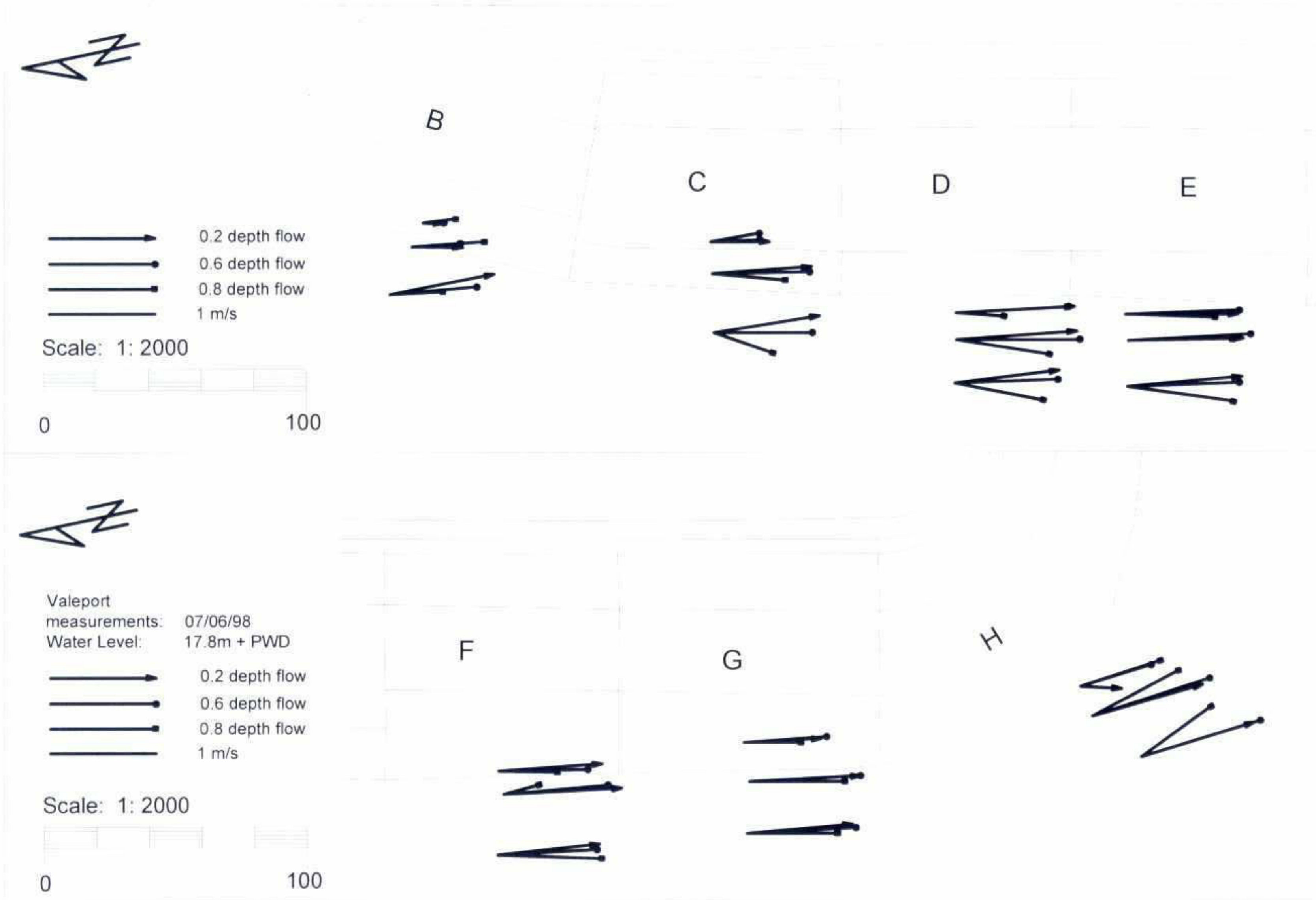


Fig. 4.4-6: Current point measurement in June 1998

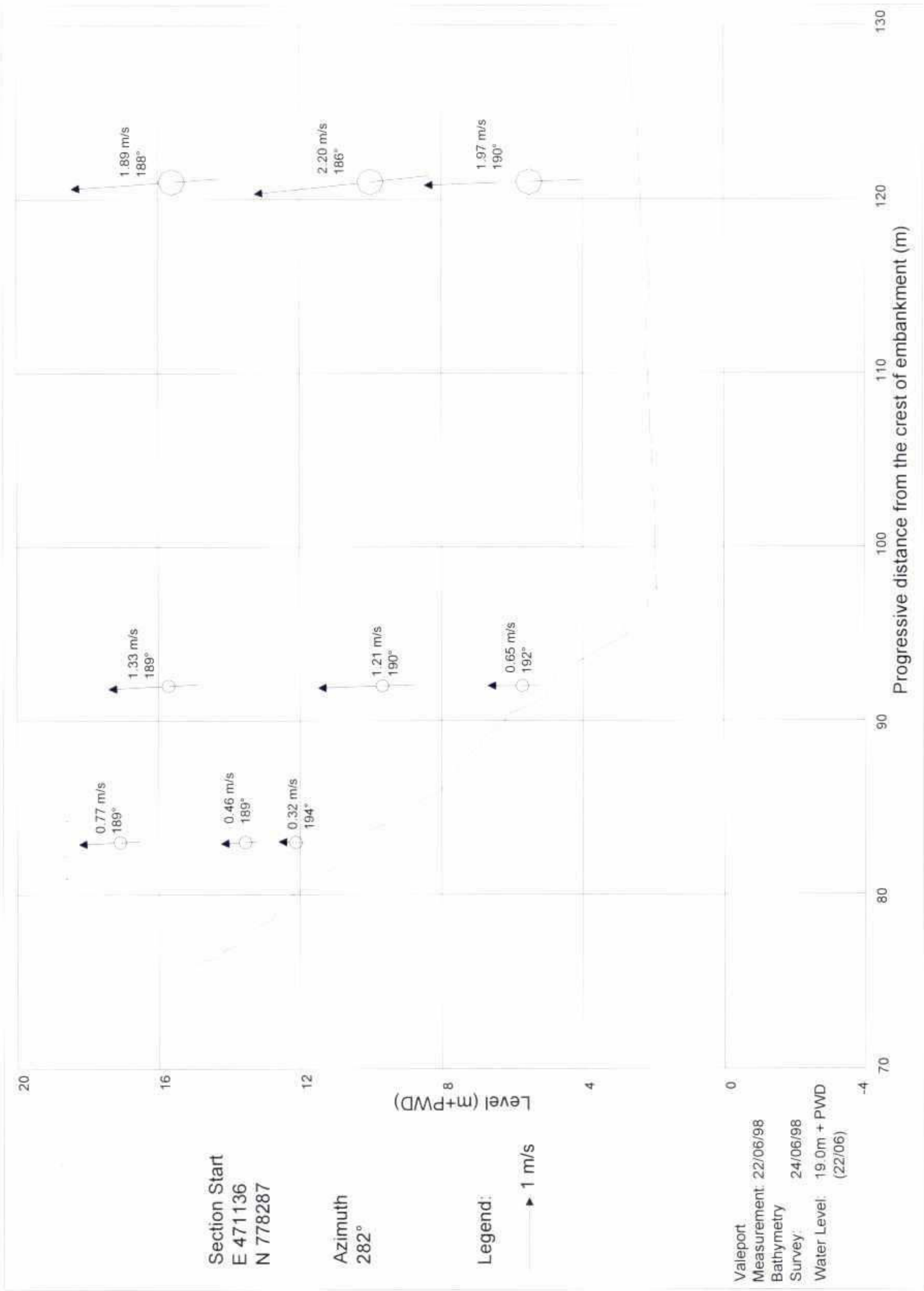


Fig. 4.4-7: Current point measurement at Section E-1

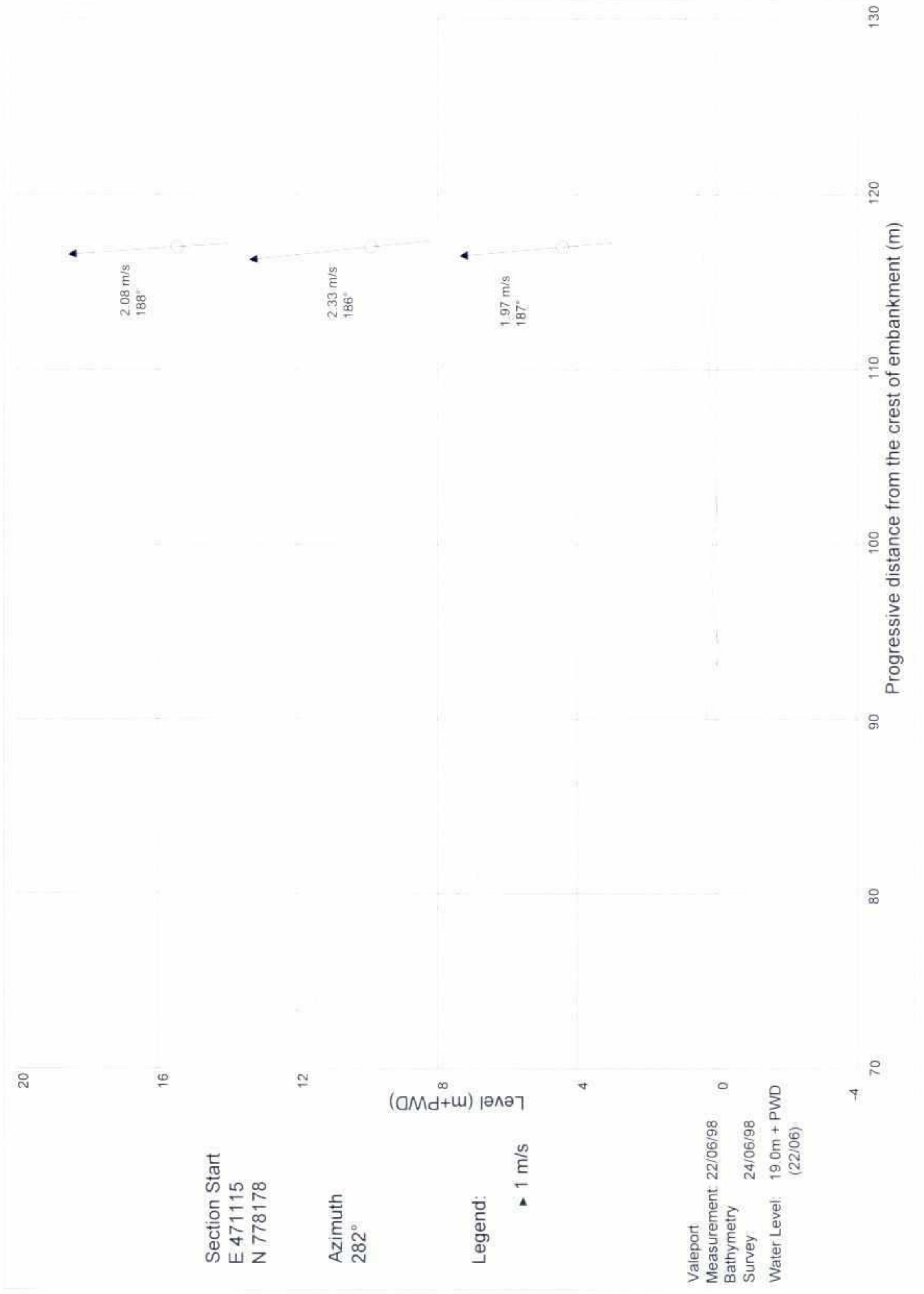


Fig. 4.4-8: Current point measurement at Section F

5 APRON INVESTIGATIONS

5.1 SIDE SCAN SONAR RESULTS

Side Scan Sonar Surveys were carried out in front of the test structure in March 1998 and in March 1999. Since in March 1998 the riverbed levels were still about –6 to –10 m+PWD, the cc-blocks of the falling aprons were only partly covered by sediments.

The blocks were detected by the Side Scan Sonar as strong reflection from the riverbed. On the slope of the falling apron the side scan sonar records indicated homogenous strong reflectors whereas in the channel the strong reflections were scattered due to partly coverage by sediments. The detected areas of strong reflection are presented in Fig. 5.5-1. Below the area presentation a photo reduced side scan sonar record (Range 50 m) of the starboard transducer is shown. From this record the launched part of Section C can be recognised as well. The detected area of that part is 32 m x 8 m.

The protrusion of the falling aprons is recorded as 25-30 m from the original end of falling aprons, which is in Section D and E almost a 200 % extension of the original width of the aprons.

The downstream area of Section E, defined as E-2, showed less strong reflectors as geosand containers were dumped at that place, but scattered strong reflectors were detected in the channel. These might be indicated by cc-blocks, which were moved downstream from the upper sections. Stronger reflections of the falling aprons in Section B and C could not be detected as these aprons consisted only of geosand containers.

Another side scan sonar inspection was done in March 1999 (Fig. 5.1-2). Since the riverbed at that time was between 0 to 5 m+PWD due to sedimentation, only the slope of the falling aprons could be investigated.

The upstream part of the launching apron at Section C continued to be launched as it was recorded by this survey. An area of 50 m x 12 m was detected.

A comparison of the reflection between the slopes at Section D and E-1 with the Section F to H shows a more scattered strong reflection at the latter sections, which implies more sedimentation on these slopes.

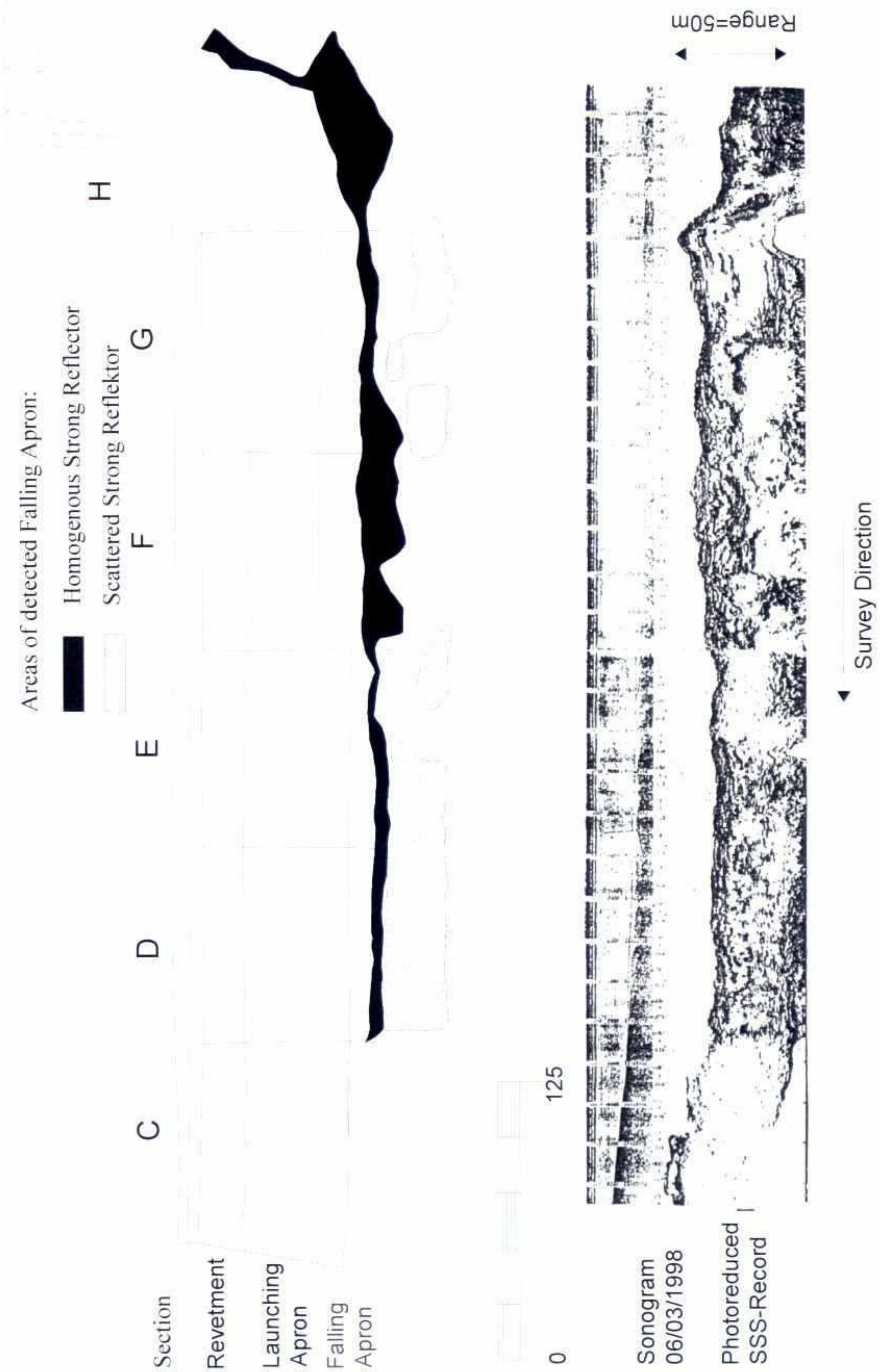


Fig. 5.1-1: Side Scan Sonar Coverage March 1998

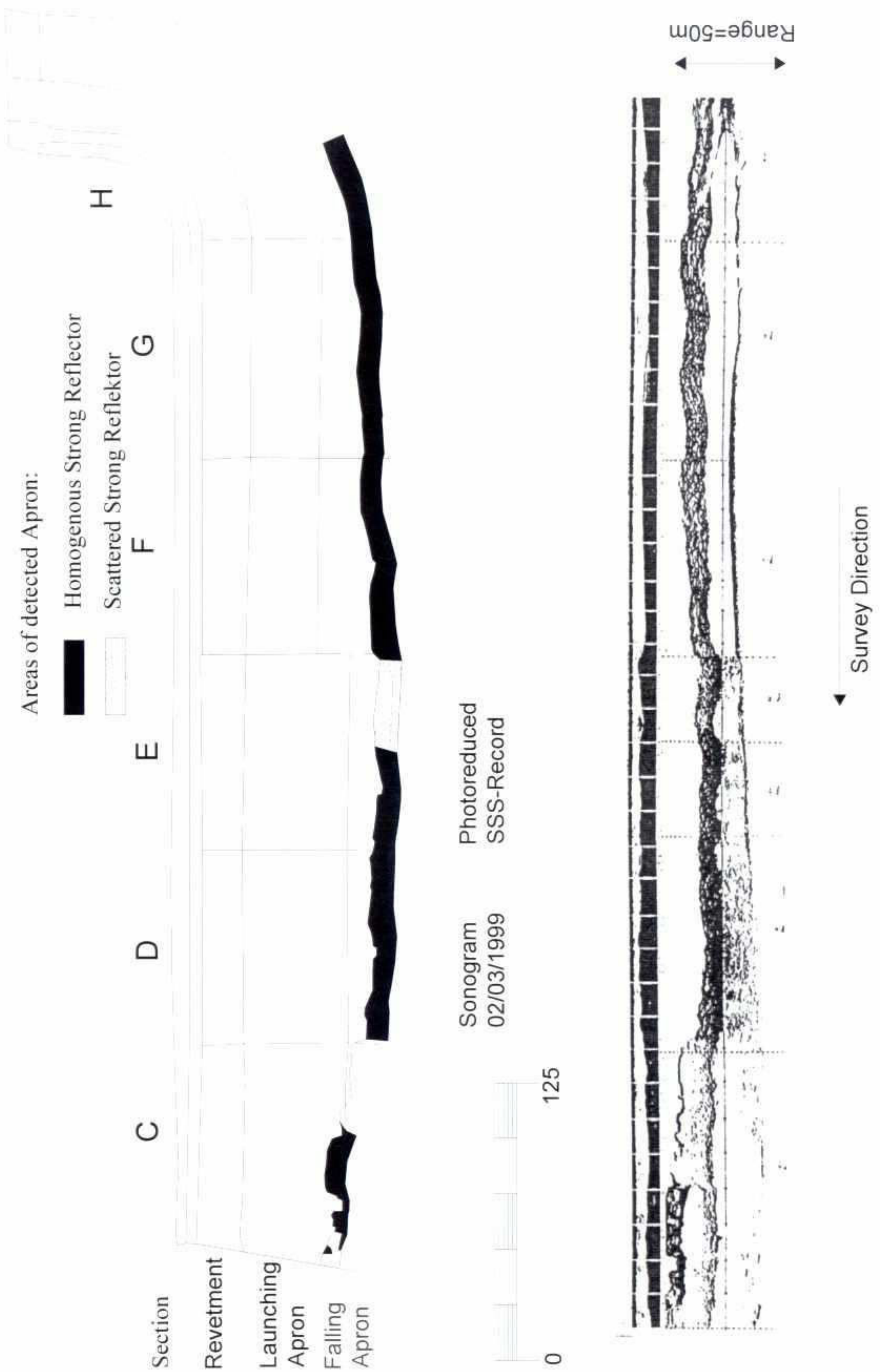


Fig. 5.1-2: Side Scan Sonar Coverage March 1999

5.2 SUBBOTTOM PROFILING

5.2.1 Summary

The subbottom profiling was carried out in combination with the side scan sonar survey in March 1998. The profiles were sailed across the river channel towards the structure. The profile separation was chosen as 10 m.

The intention of this survey was to detect material from the falling aprons, which could not be detected by the side scan sonar due to coverage by sediment. The subbottom profiles at the centre of each section are presented and interpreted in Subsection 5.2.2.

The following table lists the locations of objects, which can be clearly identified as hyperbolic reflections.

Section	Distance from original end of falling apron [m]	Depth below river bed [m]	Figure No.
D, centre	116	0.5	5.2-3
E-2, 20 m d/s	105	1.0	5.2-11
F, centre	88	1.0	5.2-6
F, centre	68	1.0	5.2-6
F, 30 m u/s	89	0.3	5.2-12
F, 10 m u/s	88	0.3	5.2-13
F, 10 m u/s	66	2.0	5.2-13
F, 10 m u/s	60	1.0	5.2-13
F, 30 m d/s	85	1.5	5.2-14
G, 50 m u/s	73	1.4	5.2-15
G, 30 m u/s	73	1.9	5.2-16
G, 10 m d/s	70	0.7	5.2-10
H-2, centre	46	0.6	5.2-9

Table 5.2-1: Location of objects detected by the subbottom profiler

5.2.2 Results and Interpretations

The processed centre profiles are plotted in Fig. 5.2-1 to Fig. 5.2-9. The left side of the profiles is towards the Jamuna river. The right side is towards the structure. Both shot number i.e. numbering at every seismic trace and station, i.e. progressive distance from the top of the embankment are shown in the horizontal scale, whereas the distance (metres below transducer) is shown in the vertical scale.

On almost all of the profiles recorded by the subbottom profiler two different basic reflection types are visible. Reflection type 1 is characterized by deeper subbottom penetration, continuous subbottom reflectors, and a generally weaker reflection intensity. Reflection type 2 shows only limited subbottom penetration, no continuous subbottom reflectors, and a generally stronger reflection intensity.

In the vicinity of the structure, the following material may be found on or in the riverbed:

- sediment transported and deposited by the Jamuna river: mainly sandy-silty sediments or even finer grain-sizes (silty-clayey material), and
- material used during the construction: mainly different types of concrete blocks and boulders.

These two major groups of materials within the working area are generally characterised by different acoustical properties. The natural sediments delivered by the Jamuna are soft sediments, whereas the artificial materials used for the construction of the structure are hard materials. The main difference between the two types of material is the density. Soft sediments are characterised by a generally lower density than hard materials. Density is in conjunction with the speed of the main component of the so-called acoustic impedance ($I = \rho \times Vp$, where ρ = density and Vp velocity of sound). It has been found in numerous studies, that for near-surface sediments the density governs the acoustic impedance. The higher the acoustic impedance of the material on the riverbed, the more energy is reflected to a receiver, and the less energy may penetrate the layer.

Applying this to the two reflection types leads to the conclusion that the higher reflection intensity, less penetration, etc. of reflection type 2 is due to the appearance of the more dense artificial construction material. Reflection type 1 shows a deep penetration, lower reflection energy, and higher subbottom penetration. These features are the result of the lower density materials deposited on the river bed of the Jamuna.

Some of the hyperbolic echoes in the deep parts in front of the steep slope in front of the test site on the seismograms may be caused by blocks of the construction material.

Strong subbottom reflectors at the toe of the steep slope of the construction may very well be caused by construction material transported downslope.

Additional selected seismograms are presented hereafter (Fig. 5.2-10 to Fig. 5.2-16) in order to show the variation within each of the sections.

96

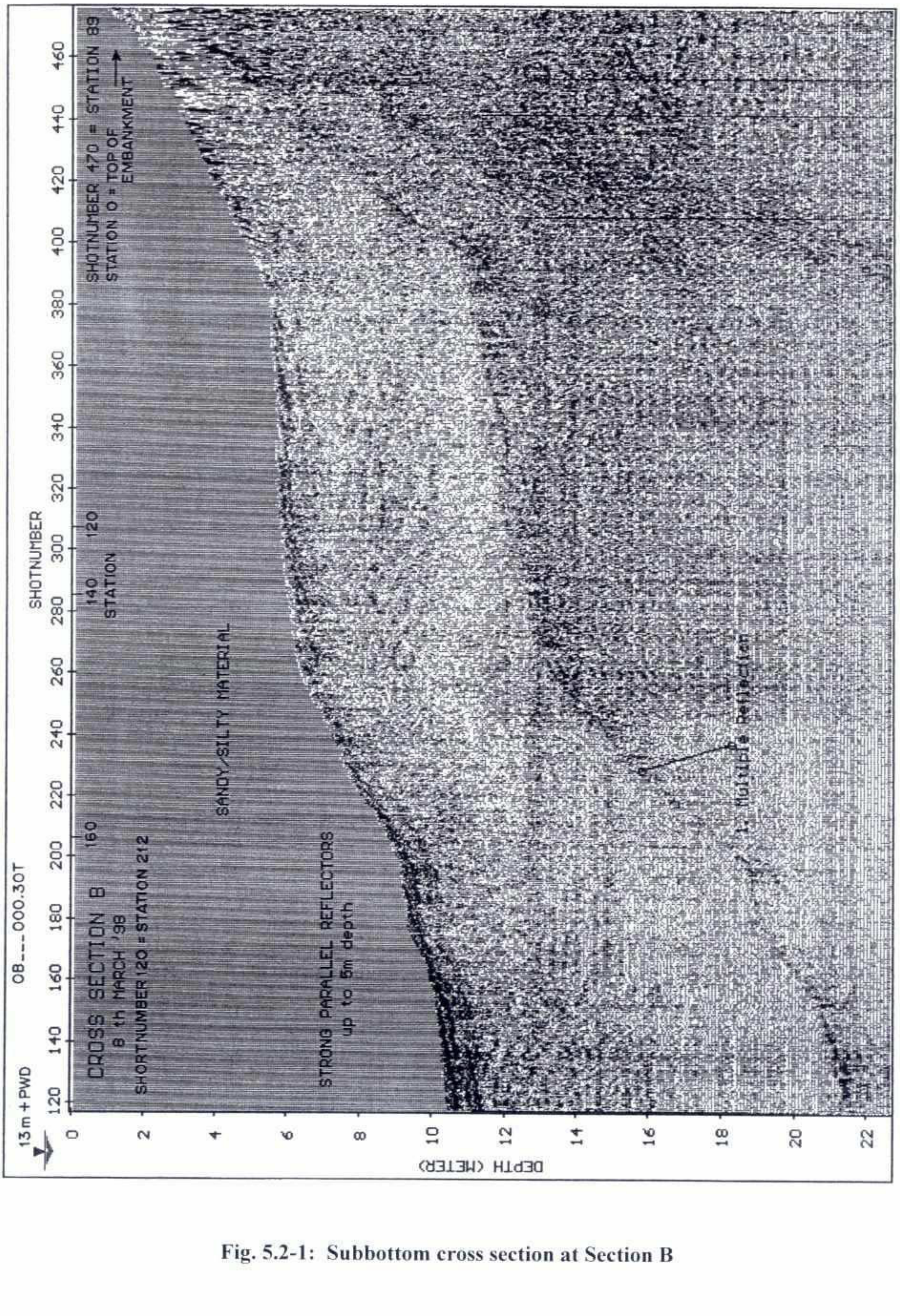


Fig. 5.2-1: Subbottom cross section at Section B

98

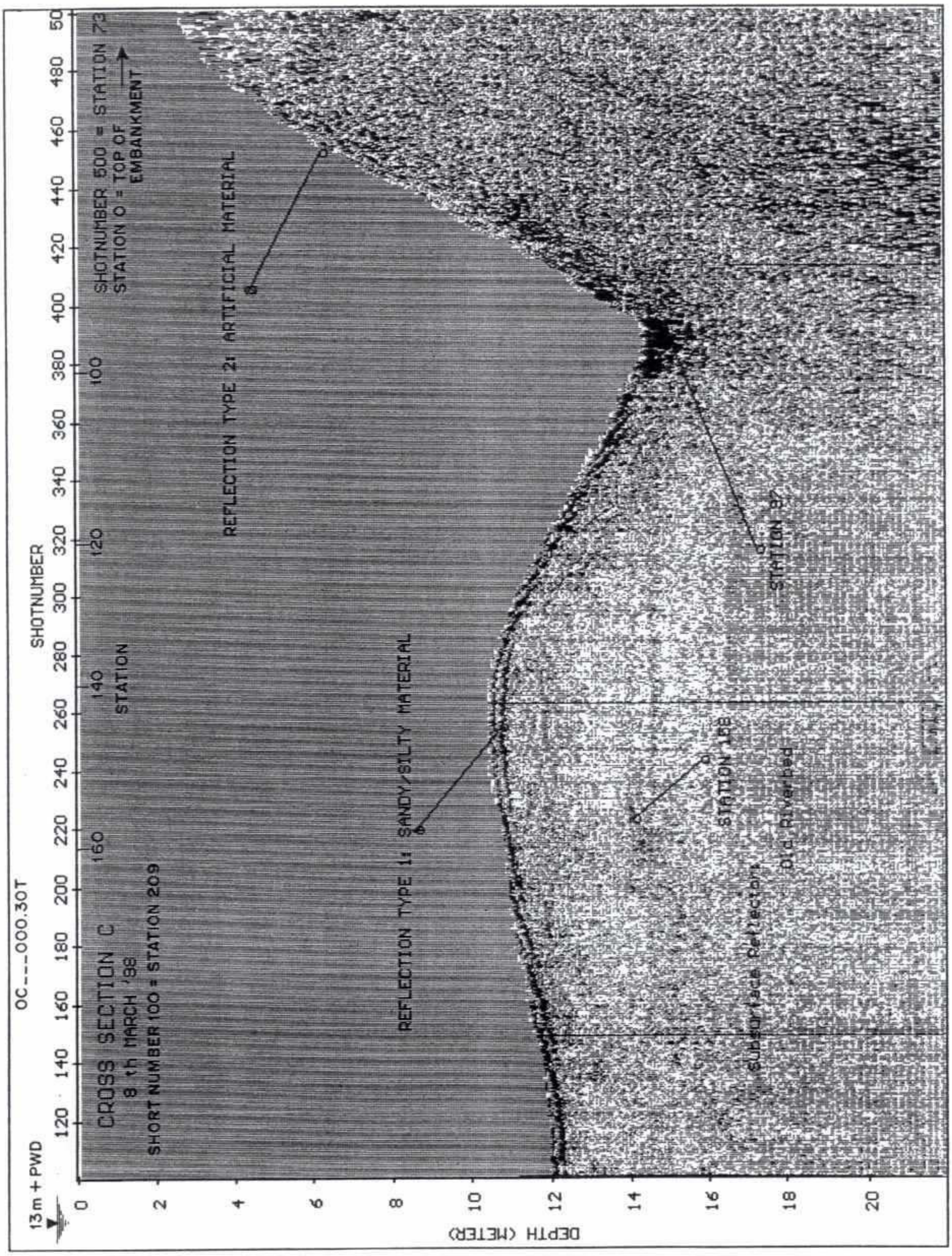


Fig. 5.2-2: Subbottom cross section at Section C

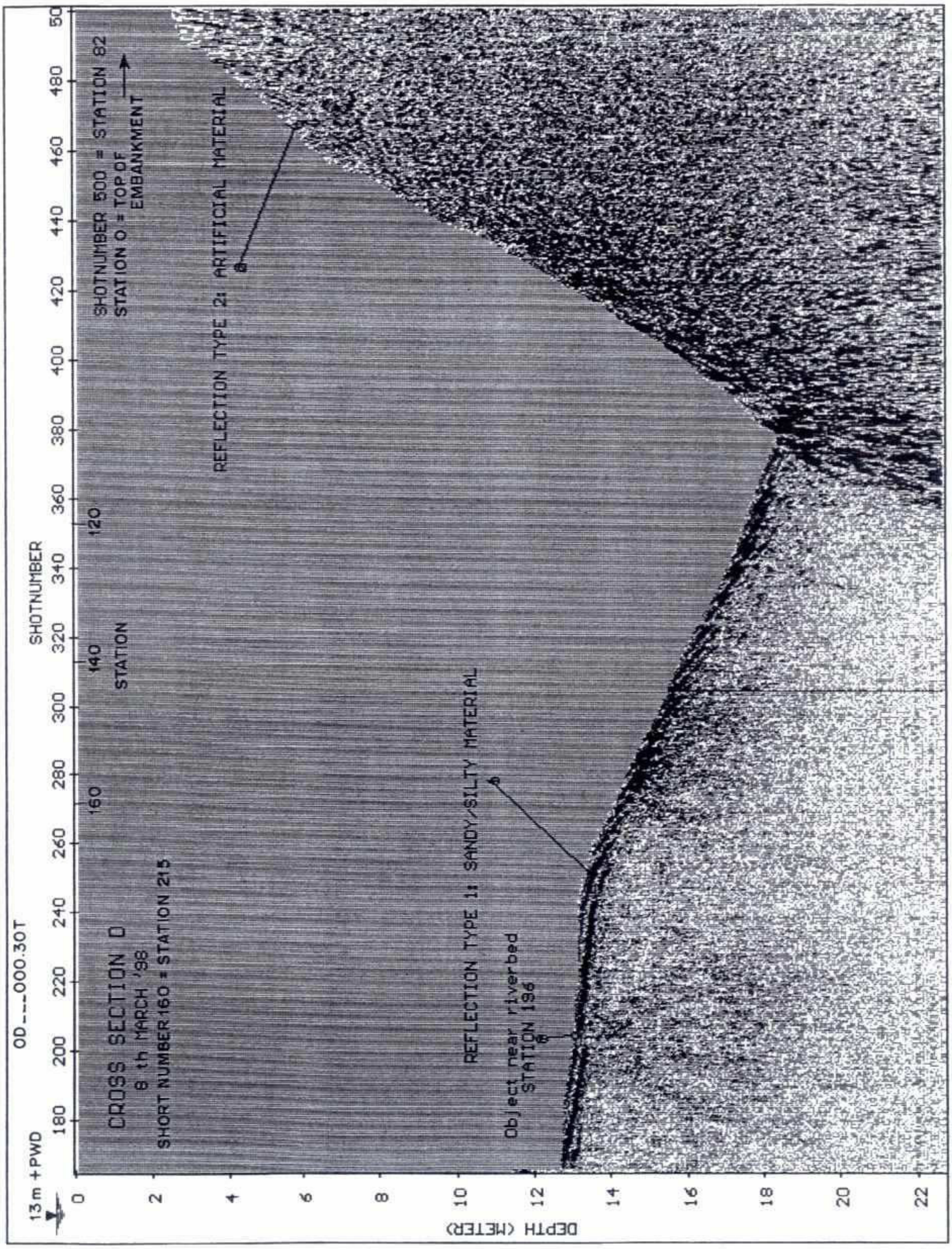


Fig. 5.2-3: Subbottom cross section at Section D

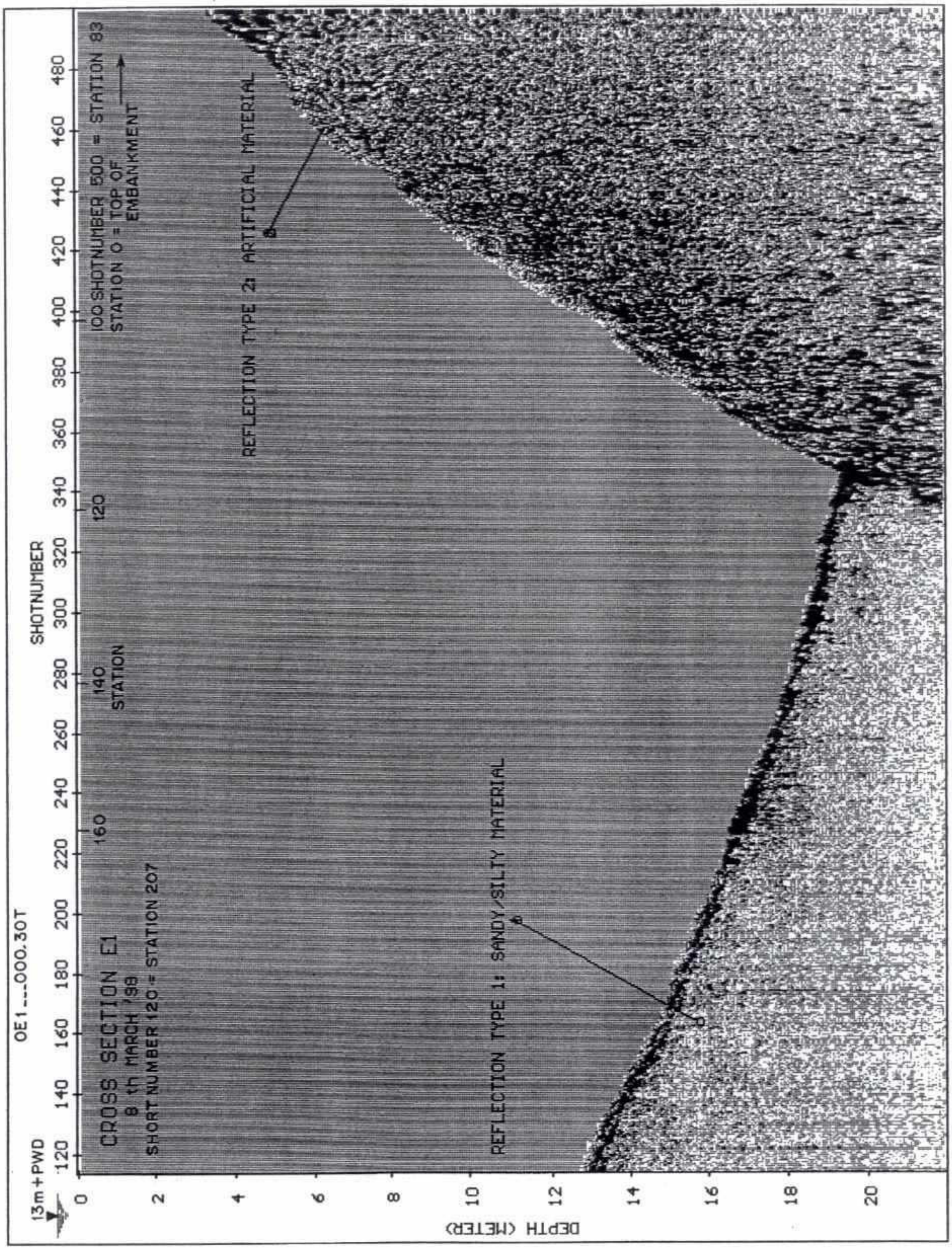


Fig. 5.2-4: Subbottom cross section at Section E1

99

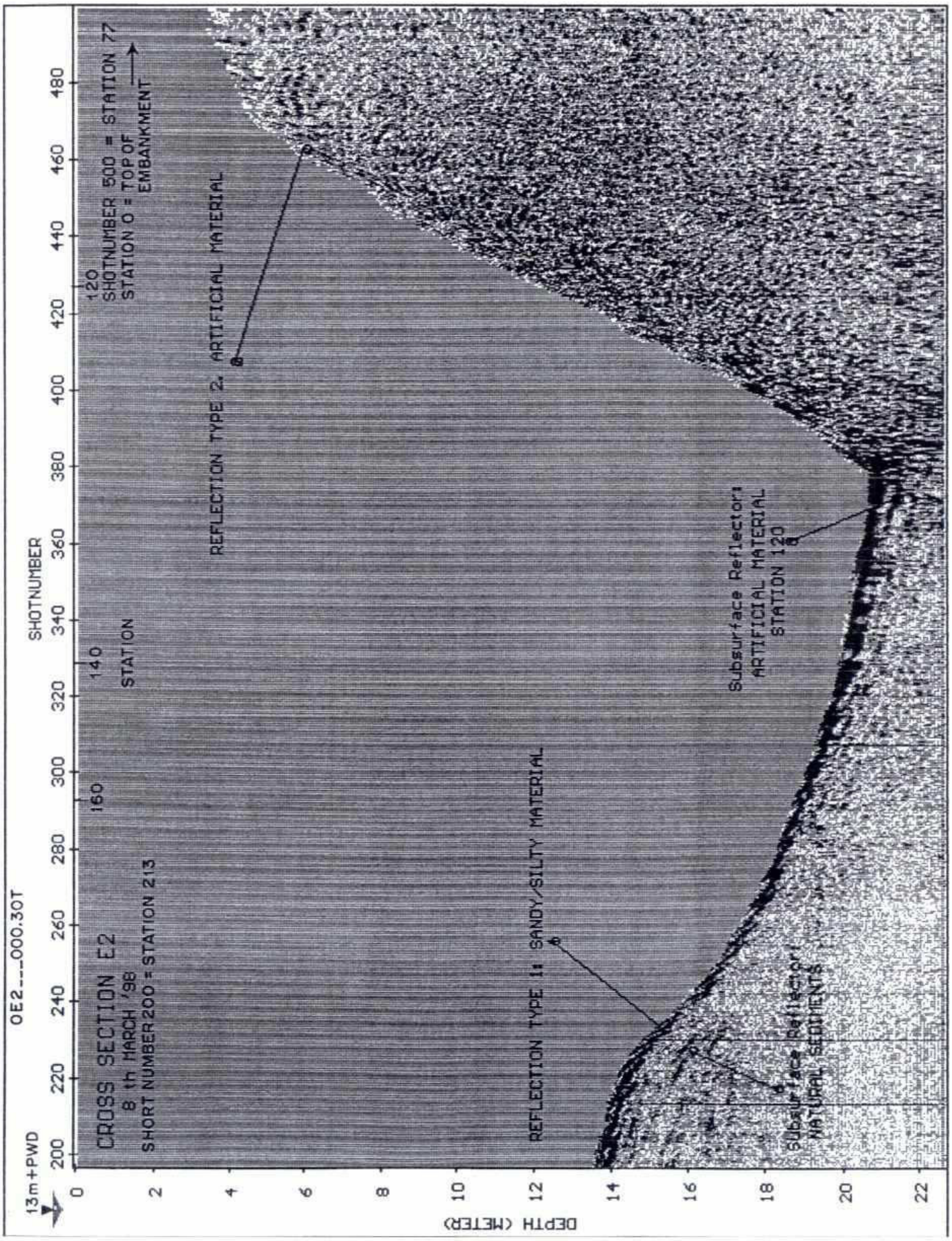


Fig. 5.2-5: Subbottom cross section at Section E2

9 +

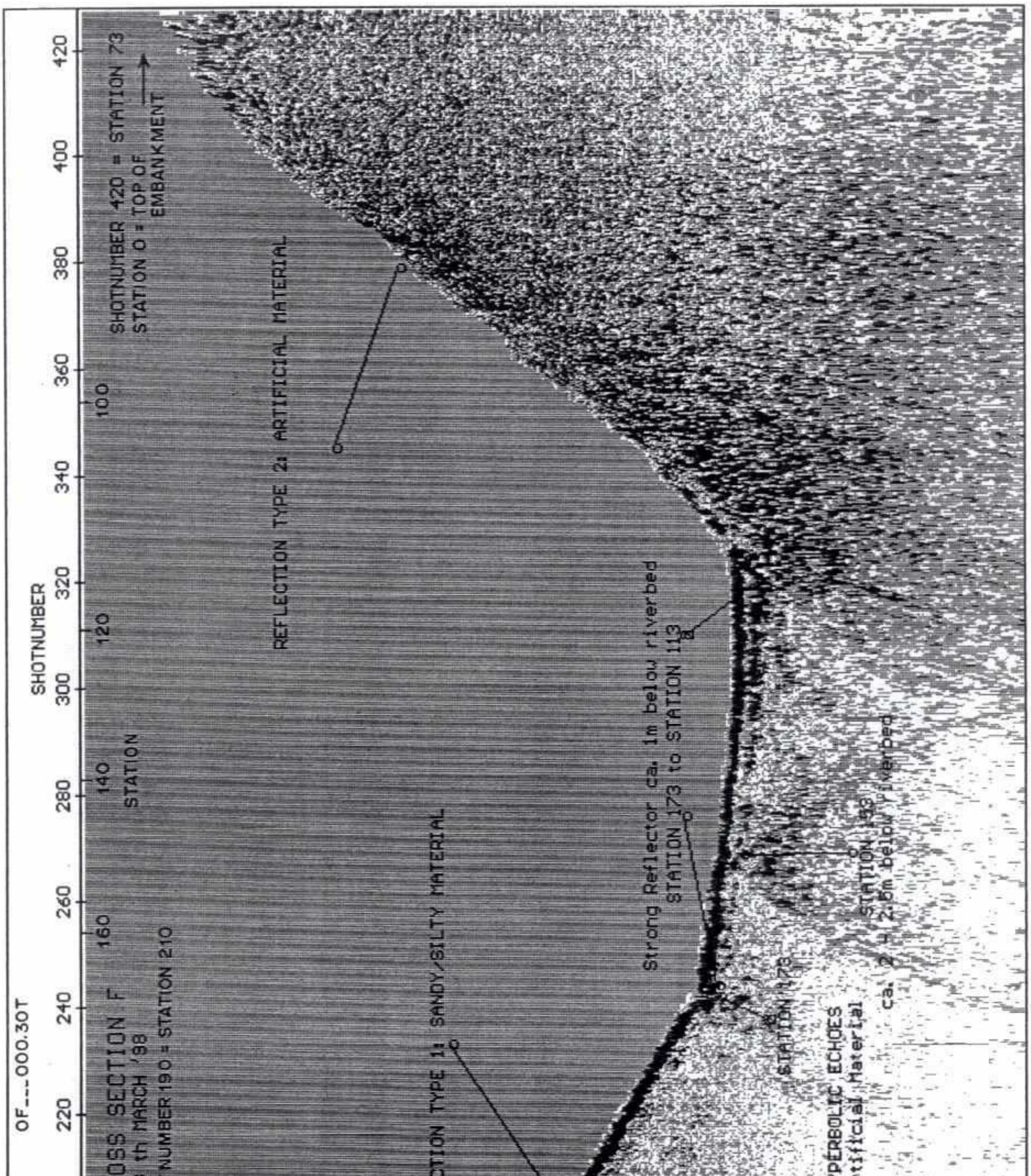


Fig. 5.2-6: Subbottom cross section at Section F

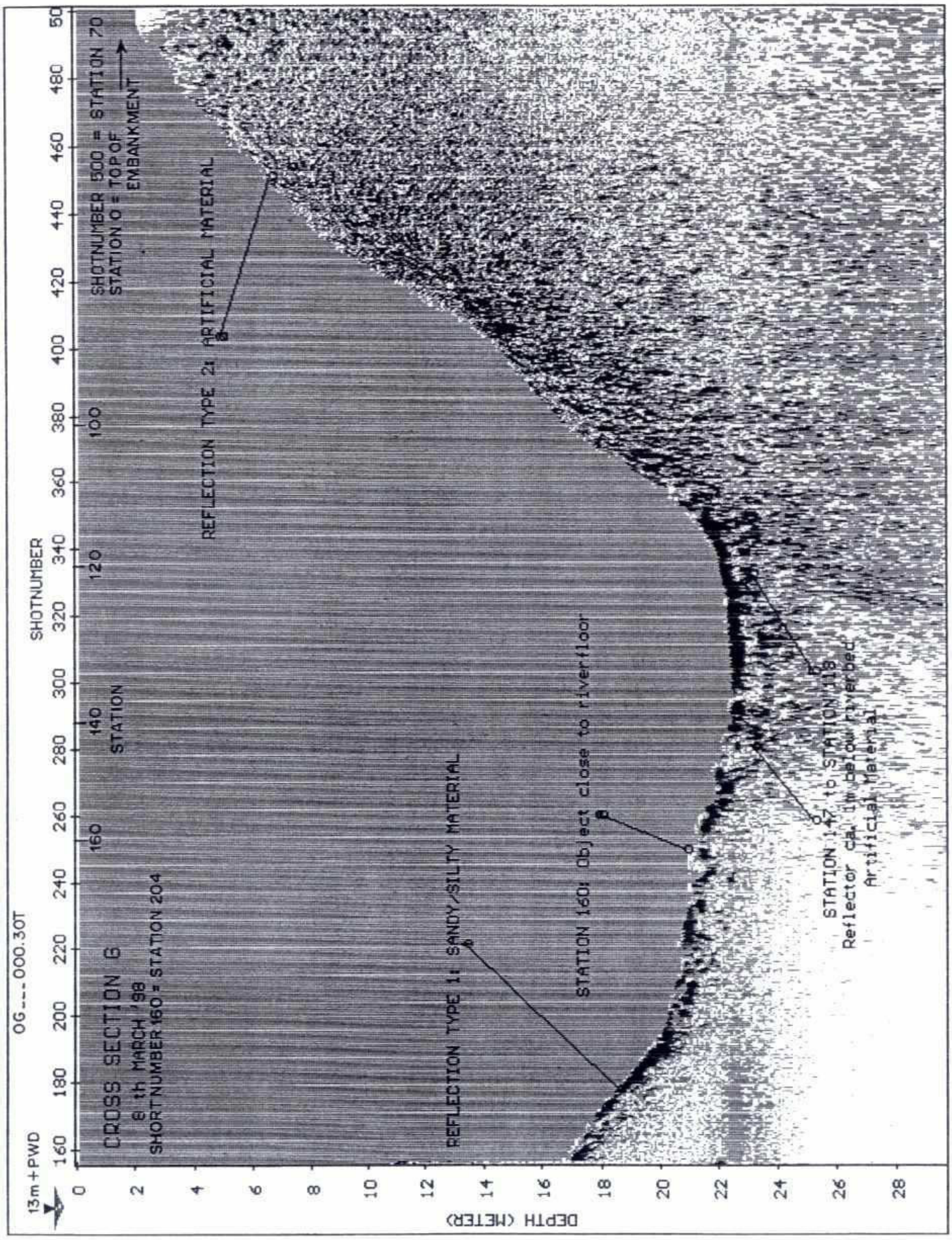


Fig. 5.2-7: Subbottom cross section at Section G

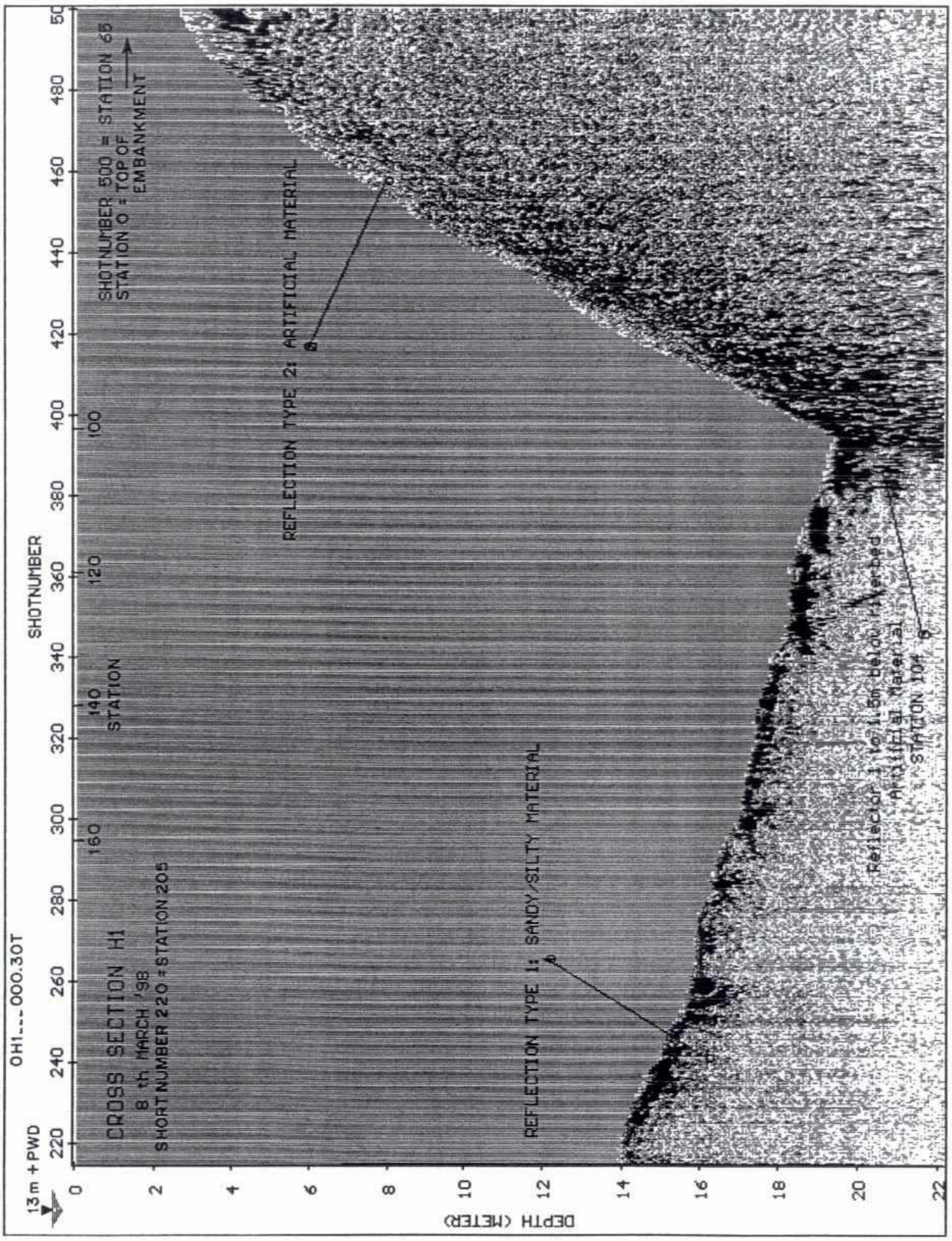


Fig. 5.2-8: Subbottom cross section at Section H1

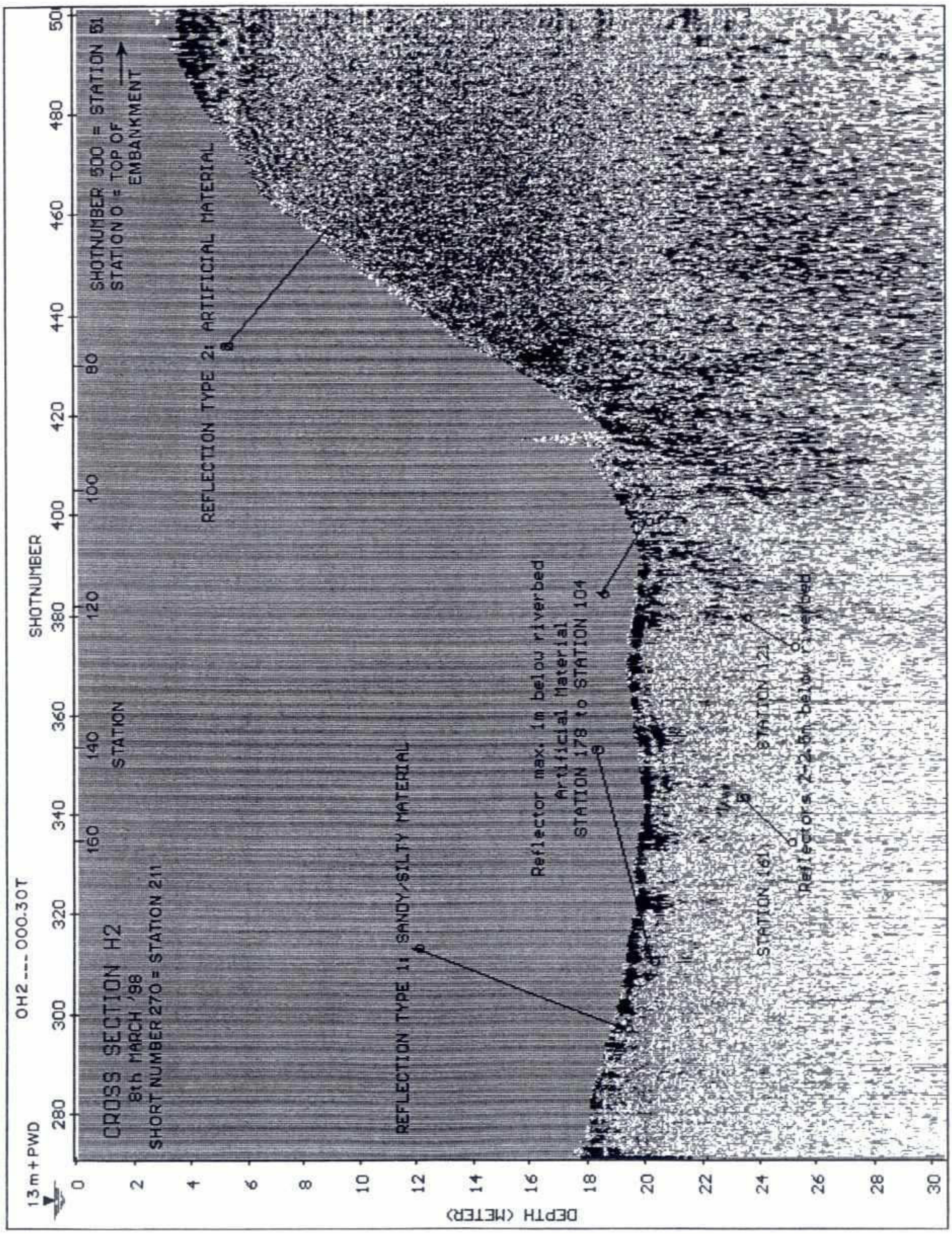


Fig. 5.2-9: Subbottom cross section at Section H2

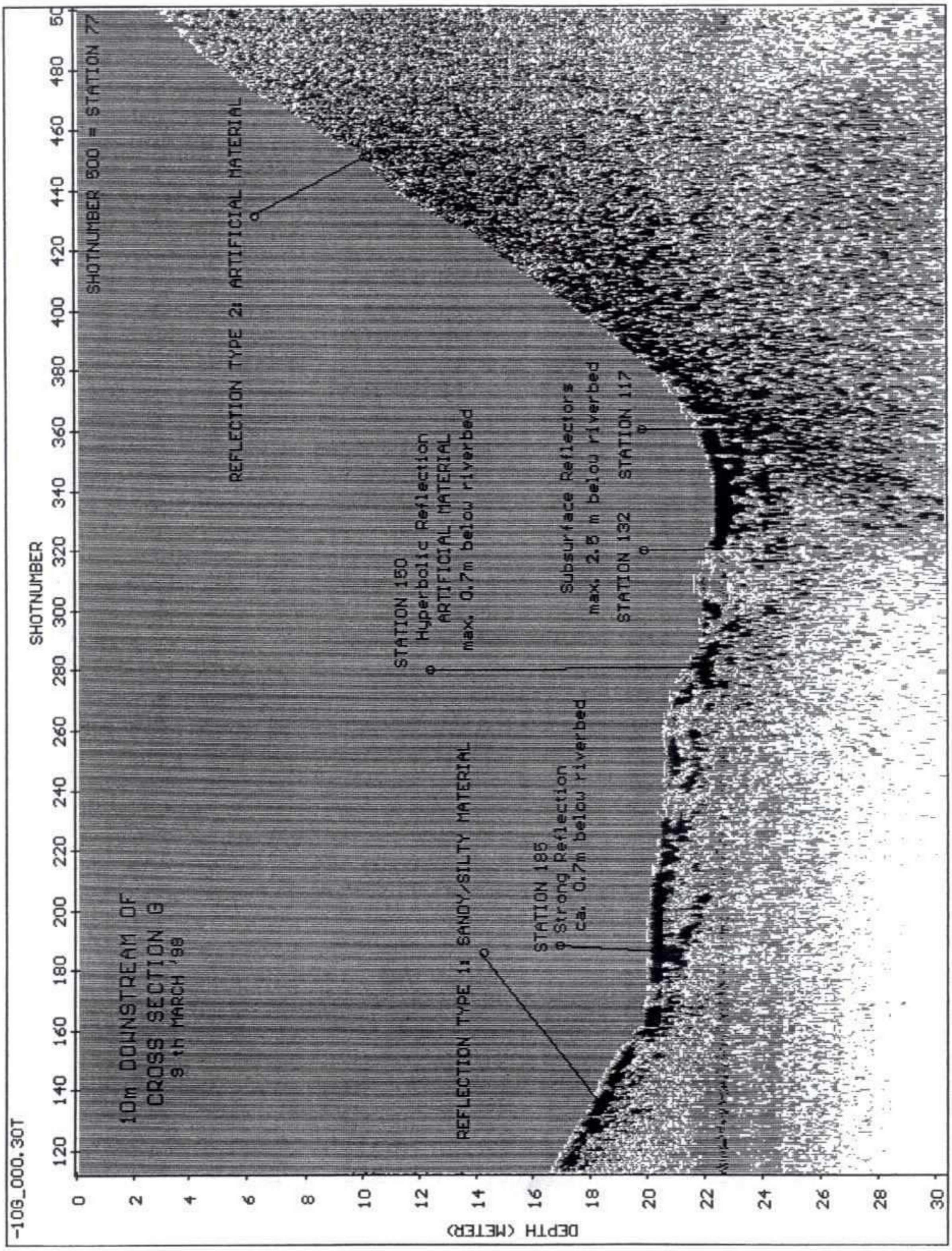


Fig. 5.2-10: Subbottom cross section 10 m d/s from Section G

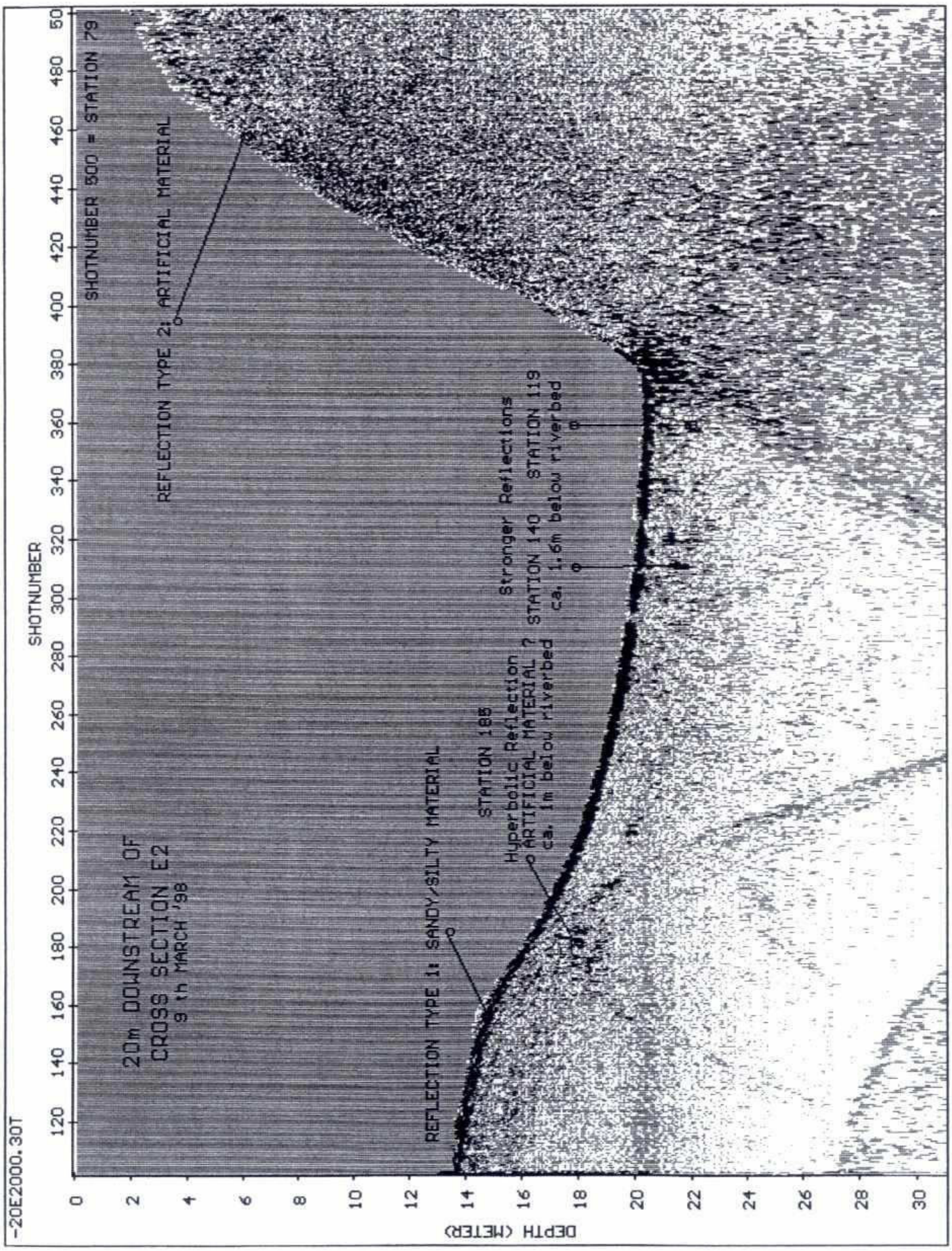


Fig. 5.2-11: Subbottom cross section 20 m d/s from Section E-2

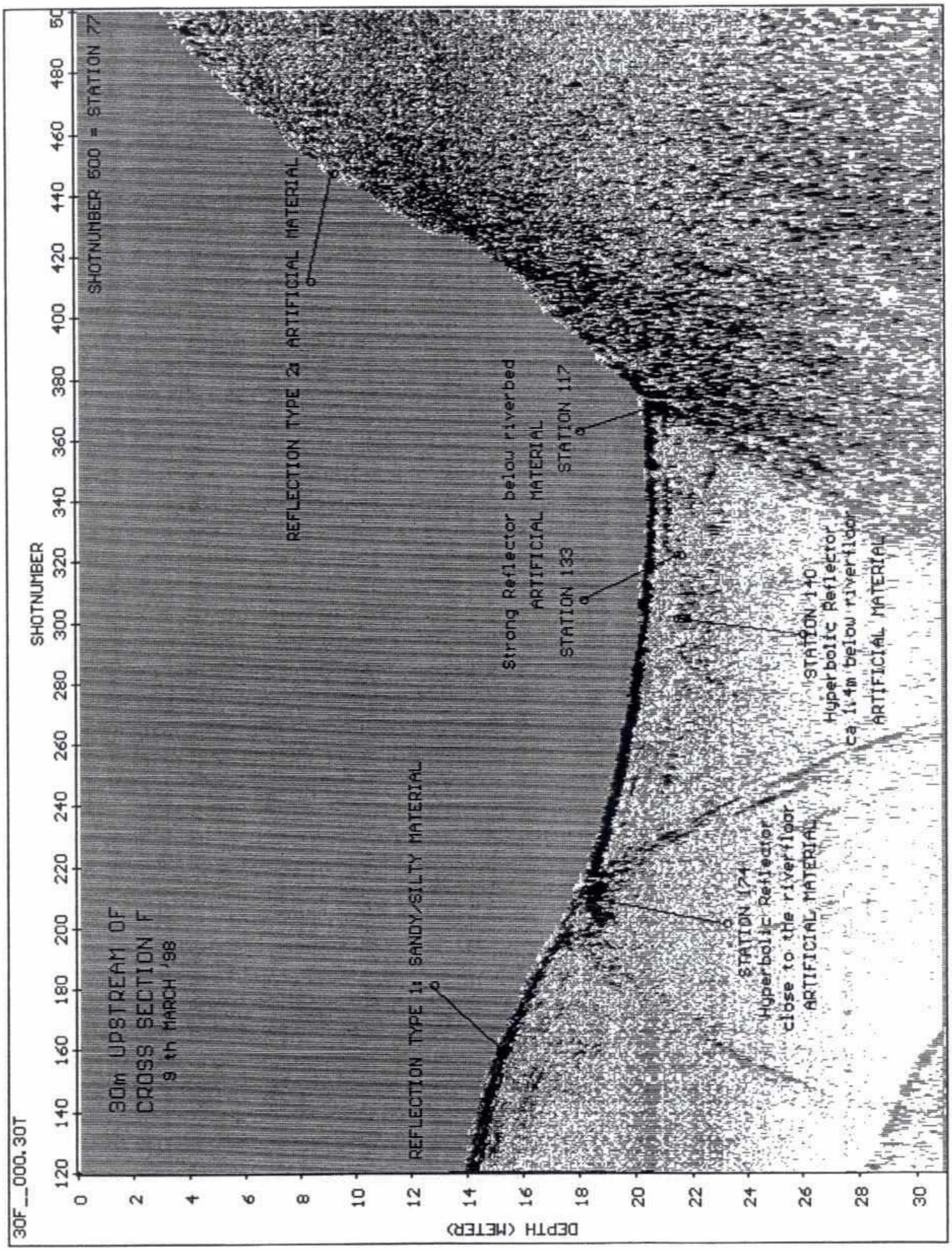


Fig. 5.2-12: Subbottom cross section 30 m u/s from Section F

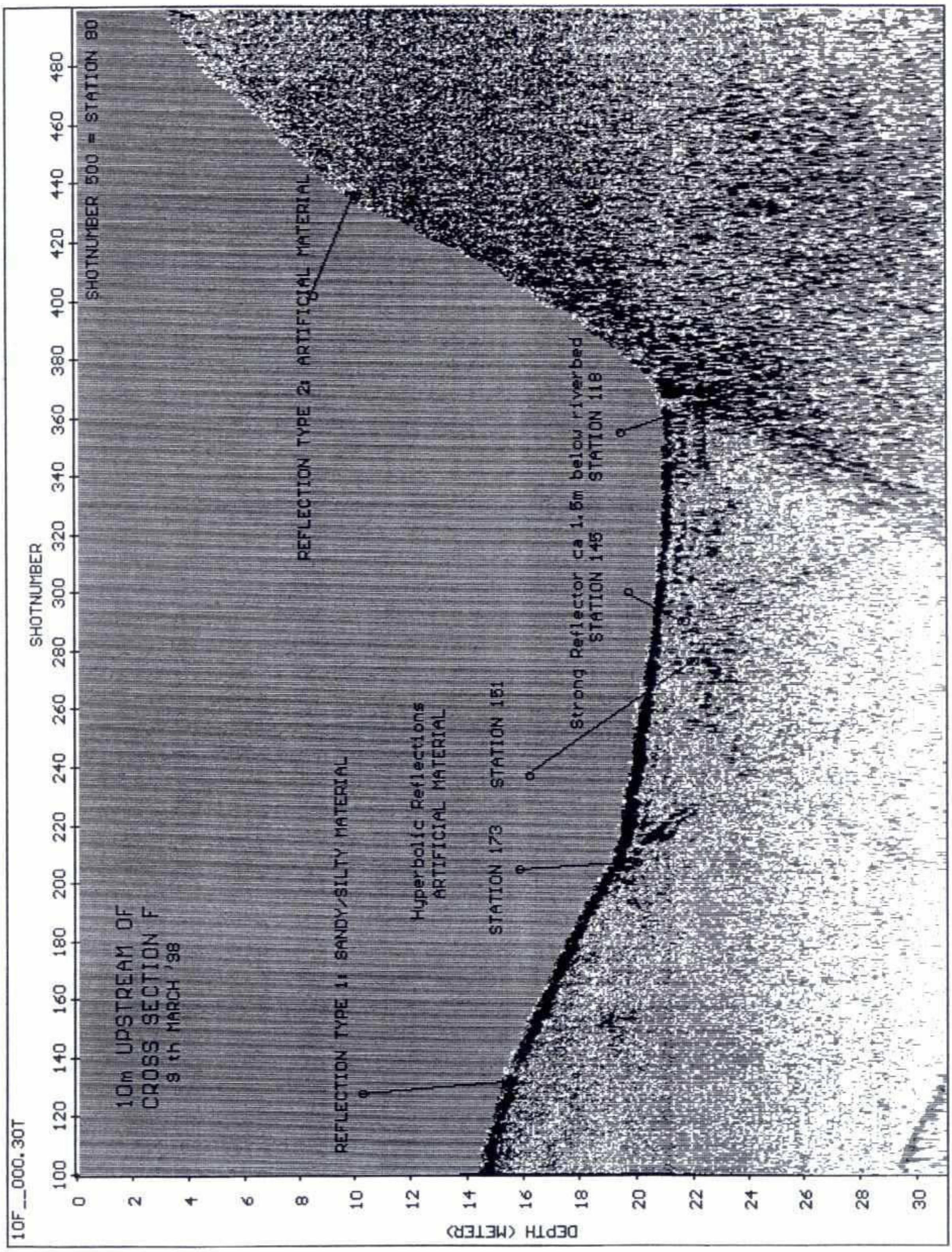


Fig. 5.2-13: Subbottom cross section 10 m u/s from Section F

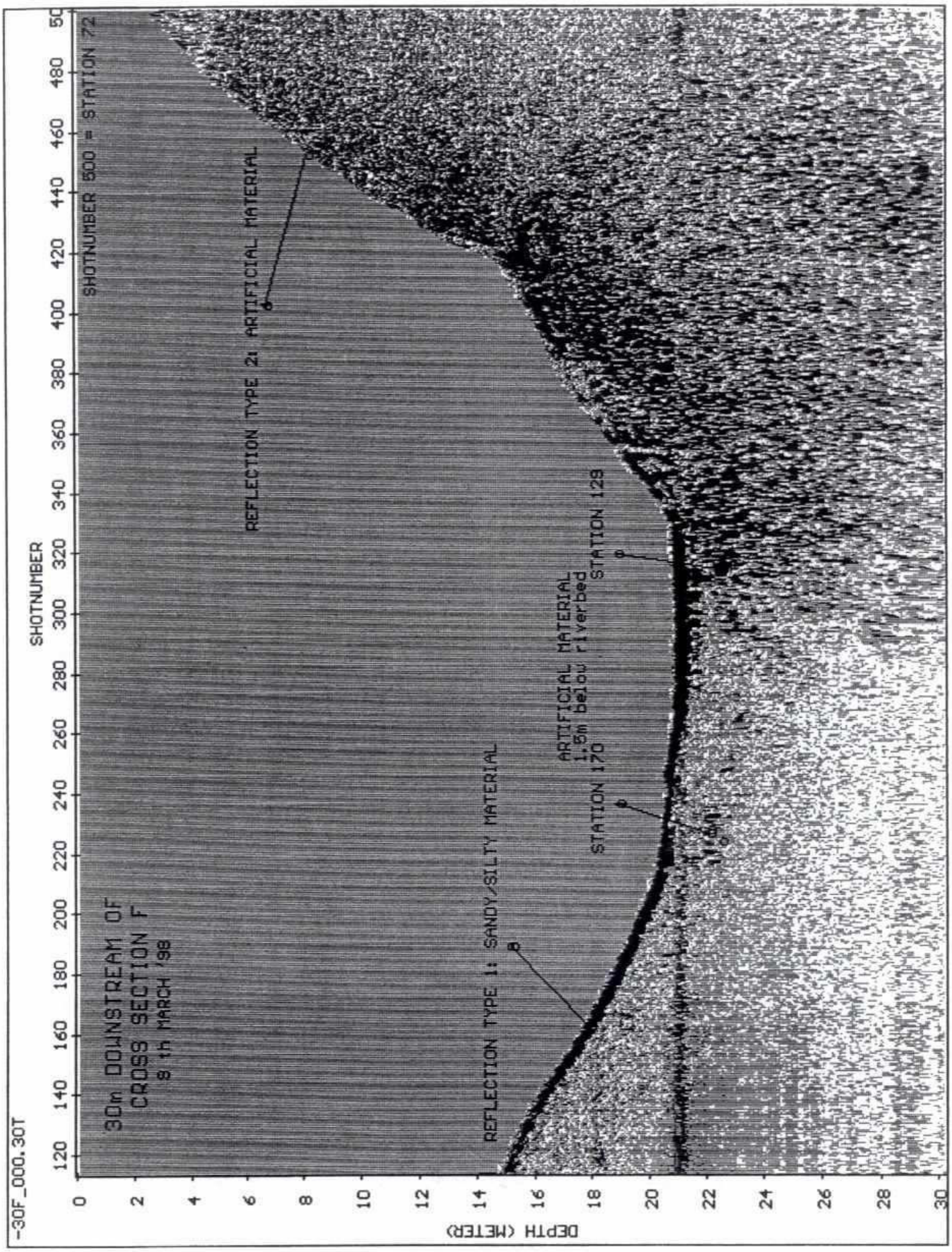


Fig. 5.2-14: Subbottom cross section 30 m d/s from Section F

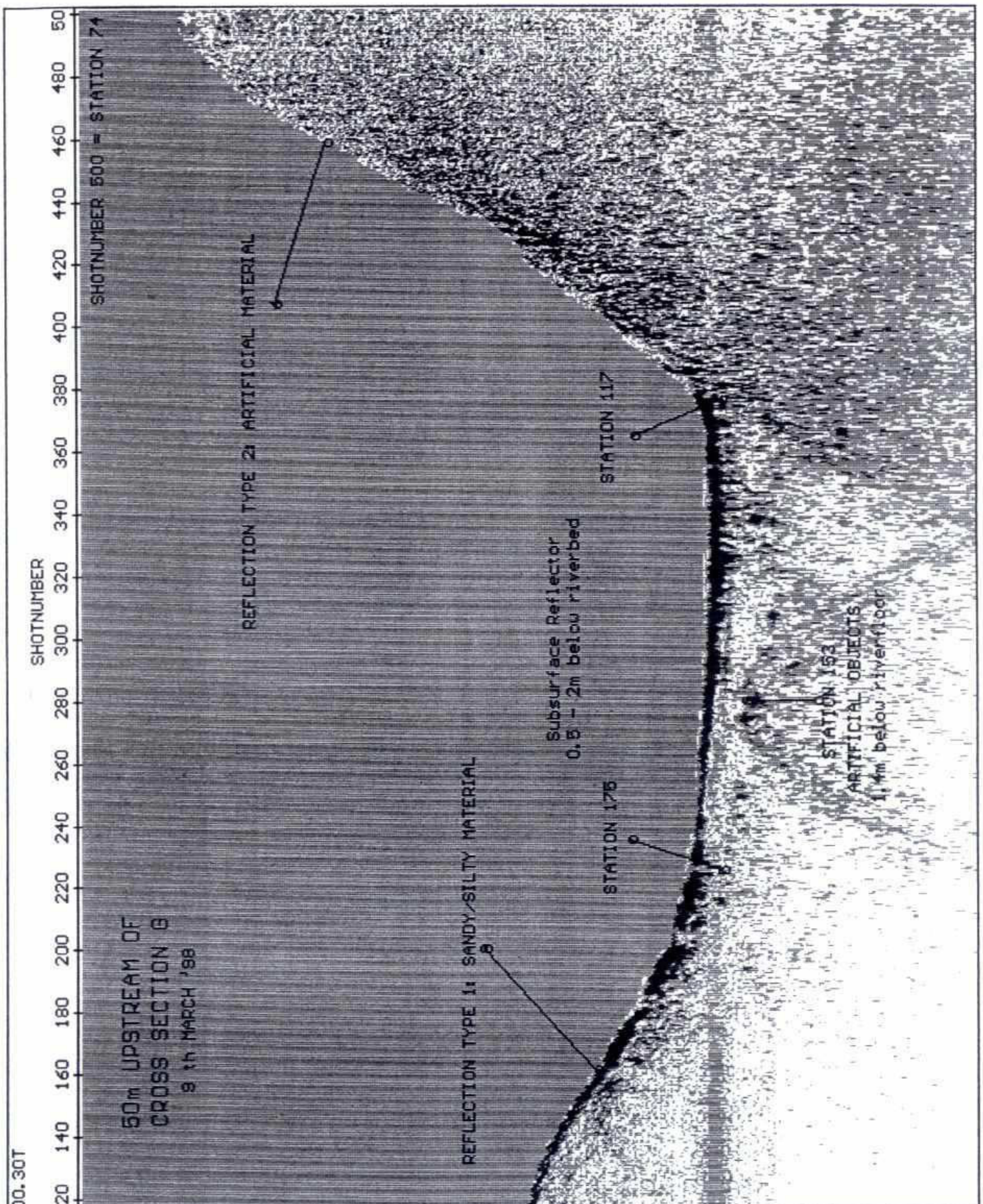


Fig. 5.2-15: Subbottom cross section 50 m u/s from Section G

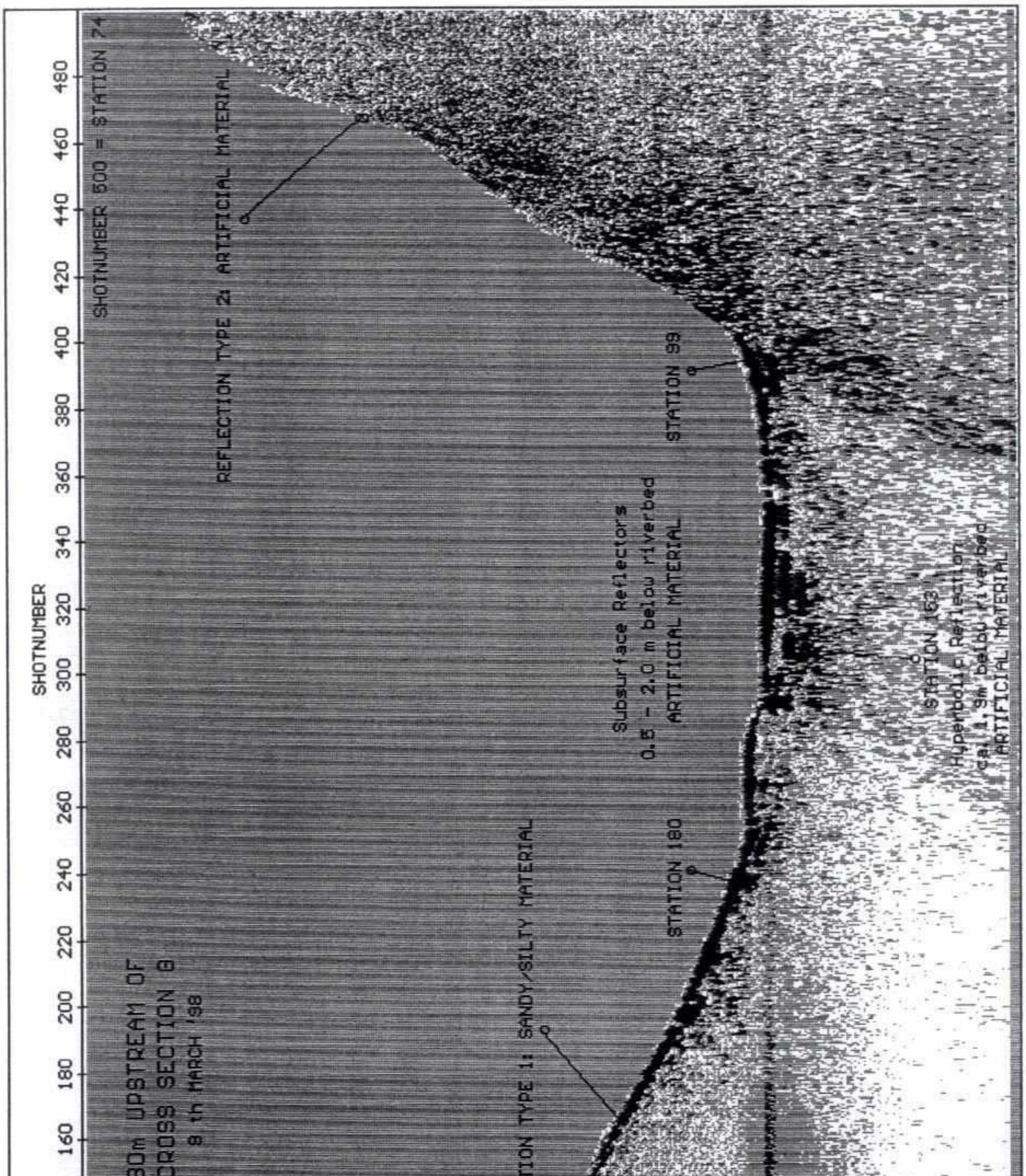


Fig. 5.2-16: Subbottom cross section 30 m u/s from Section G

5.3 DIVING INVESTIGATIONS

On February 16 and 17, 1999 the conditions of the submerged aprons were investigated by diving operations. The results and findings are described in the following. A graphical presentation in combination with the actual bathymetric cross section is shown in Fig. 5.3-1 to Fig. 5.3-6.

Transition of Section A and B:
CC-blocks were found approx. 10 m from the waterline. The blocks were scattered and did not form the slope, nor in evenline. The slope was covered by thick deposit of clayey loam at the beginning of Section B. Slightly upstream from the section the beginning of a scour hole was located. The dimension of the hole was approximately 3 m x 3 m x 1.5 m at a water depth of 4 m.

Geo-sand containers were found in Section B completely covering the bed material up to 15 m from the waterline. The bags were intact, undulating in a slope. Scattered cc-bocks were also found in this section up to 24 m from waterline. The rest of the slope was composed of soil and hard clayey soil lumps.

Mid Section of B:
Approx. 12 m from the waterline the bed was found in uniform gentle slope followed by a sudden drop, and then again uniform slope. Up to 12 m, Geo-sand container covered the slope more or less completely. After the drop, no more geo-sand container was found – the slope was covered here by clayey soil.

Transition of Section B and C:
Close to the transition and up to 10 m upstream in Section B, the slope was covered by random cc-blocks and sand bags – scattered and few in numbers.

From transition to downstream in Section C, RENO-mattress in double layers was found. Geotextile sheet separating two layers of Reno-Mattress and top layer RENO-mattress were found in good condition. Bottom layer appeared rusted. Beneath RENO-mattress Geo-sand containers were found. This was located up to 14 m along the slope from the waterline.

After this 14 m, geo-sand container in uniform pattern and then scattered geo-sand container covered the further 16 m of the slope.

Upstream Transition of C:
The slope was covered by RENO-Mattress up to a distance of 12 m from the waterline. The connecting wire between adjacent first and second RENO compartment at a distance of about 3 m from the waterline was torn away and the mattress rolled upward. Gravel was partly lost from the top.

Geo-sand container covered the slope from 12 m to 36 m. They were not in uniform thickness, rather were found in single, double or even in triple layer. Between geo-sand containers, the soil surface was encountered, but these gaps were small. Deposit of silty loam type soil in dense layer was found behind the ending geo-sand container. No soil deposit was encountered over the geo-sand container, but algae type growth was detected.

Mid of Section C:
RENO-mattresses covered extended from waterline to 7 m down along the slope and was in good condition. No rupture or damage was detected. No deposit of soil was encountered over RENO. Geo-sand containers were placed from 7 m to 25 m down along the slope. The slope was not uniform. At

the end of Geo-sand containers, a vertical trench like a formation in north-south direction for a short distance was found. All the containers were intact. Loose deposit of sand was found between gaps of the containers.

Section D:
The end of the falling apron was detected at a distance of 24 m from the waterline. The cc-blocks on the falling apron built a non-uniform slope up to the end and between them coarse sand deposit was found. A sunken engine country boat was found at the end of falling apron. The pattern of the cc-blocks was the same as it was visible above the waterline.

Section E-1:
CC-blocks were found up to a distance of 12 to 15 m from the waterline, but not shown in a uniform pattern. Intermediate gaps in a distance of 2 to 3 meters were found and uncovered soil was seen in these gaps. At the end, the bed raised slightly.

Section E-2:
Scattered gabion sacks were found with intermittent gaps of 2 to 3 meters. The gaps soil cover showed only coarse sand deposit, which was extended up to 15 to 18 m from the waterline. The bed raised slightly at the end.

Section F:
CC-blocks covered a length of 13 m towards the river. The blocks did not cover completely the total length. In some places the blocks were heaped up and in other places, no single block was found. The heap was quite vertical and seemed to collapse by small disturbance. A uniform channel like a formation between blocks in longitudinal direction was found.

Section G:
Scattered blocks were found up to a distance of 15 m from the waterline. A lot of gaps between the blocks were found. Sited sand of 1 m width in longitudinal direction was found in some parts.

Section H:
The cc-blocks of the falling apron extended towards the river in a more or less uniform pattern up to a distance of 16 m to 8 m in upstream to downstream direction. Further to the river they showed a more scattered pattern. Clay soil was encountered after cc-block.

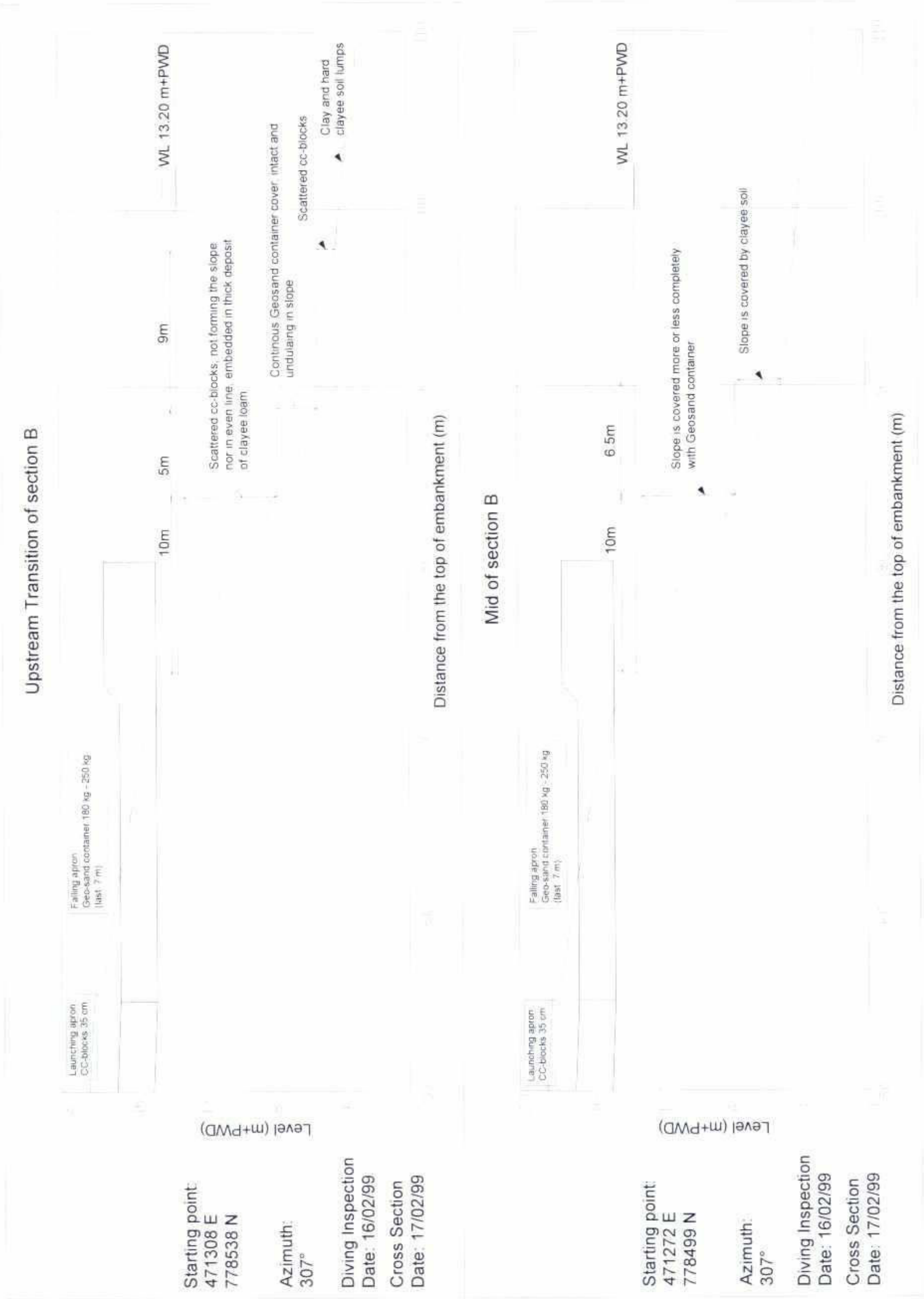


Fig. 5.3-1: Diving inspection at Section B

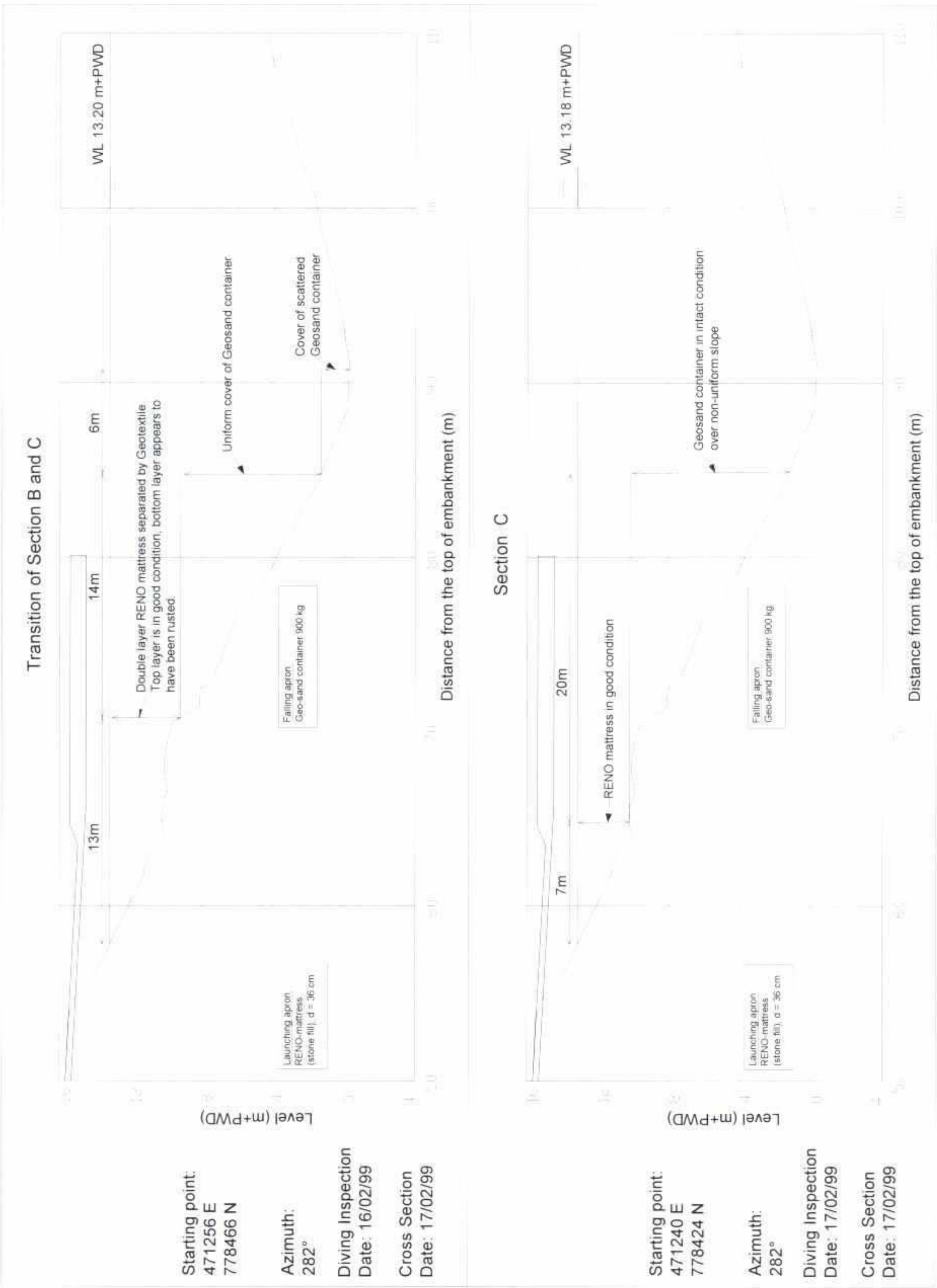


Fig. 5.3-2: Diving inspection at Section C

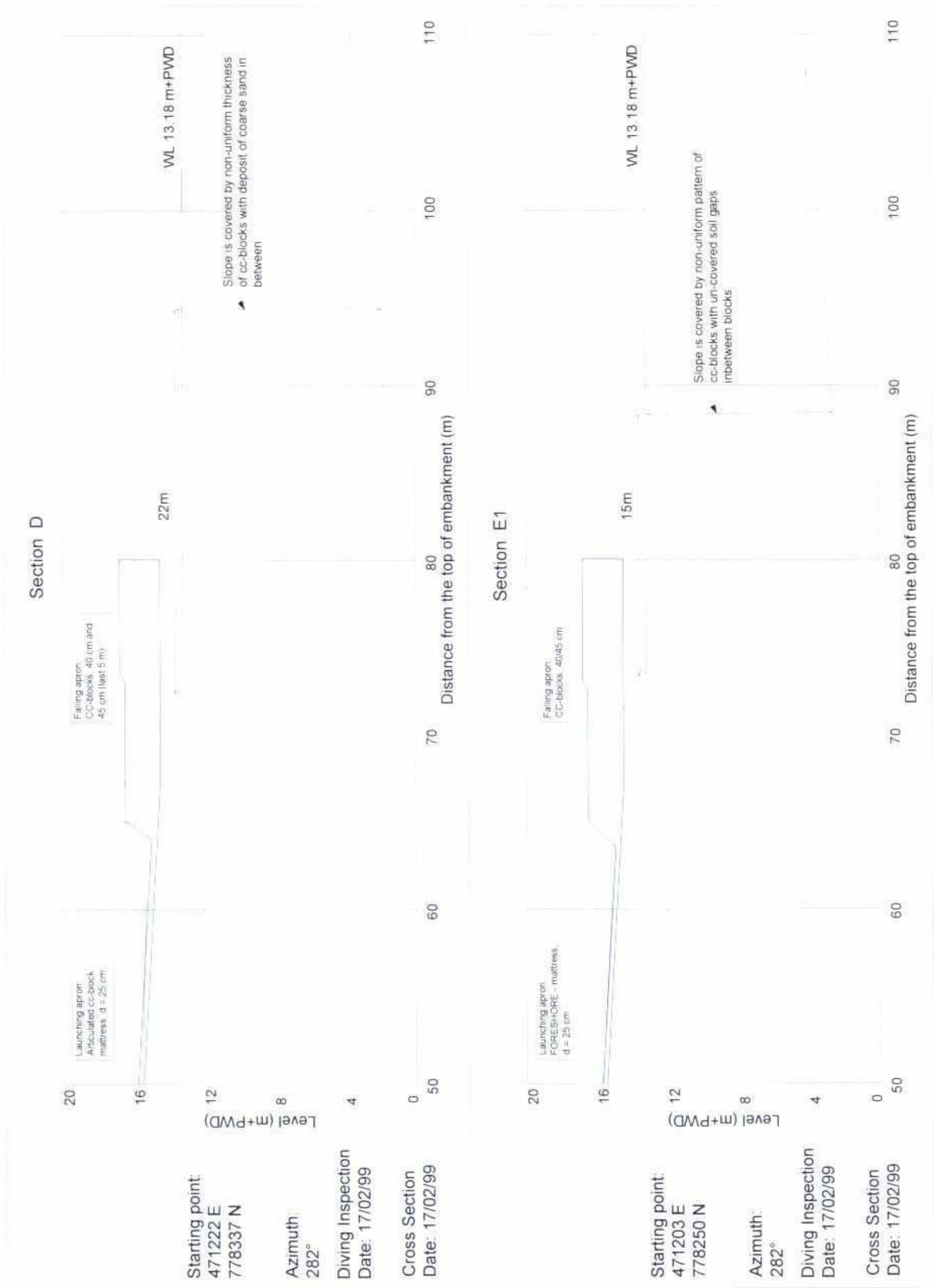


Fig. 5.3-3: Diving inspection at Sections D and E1

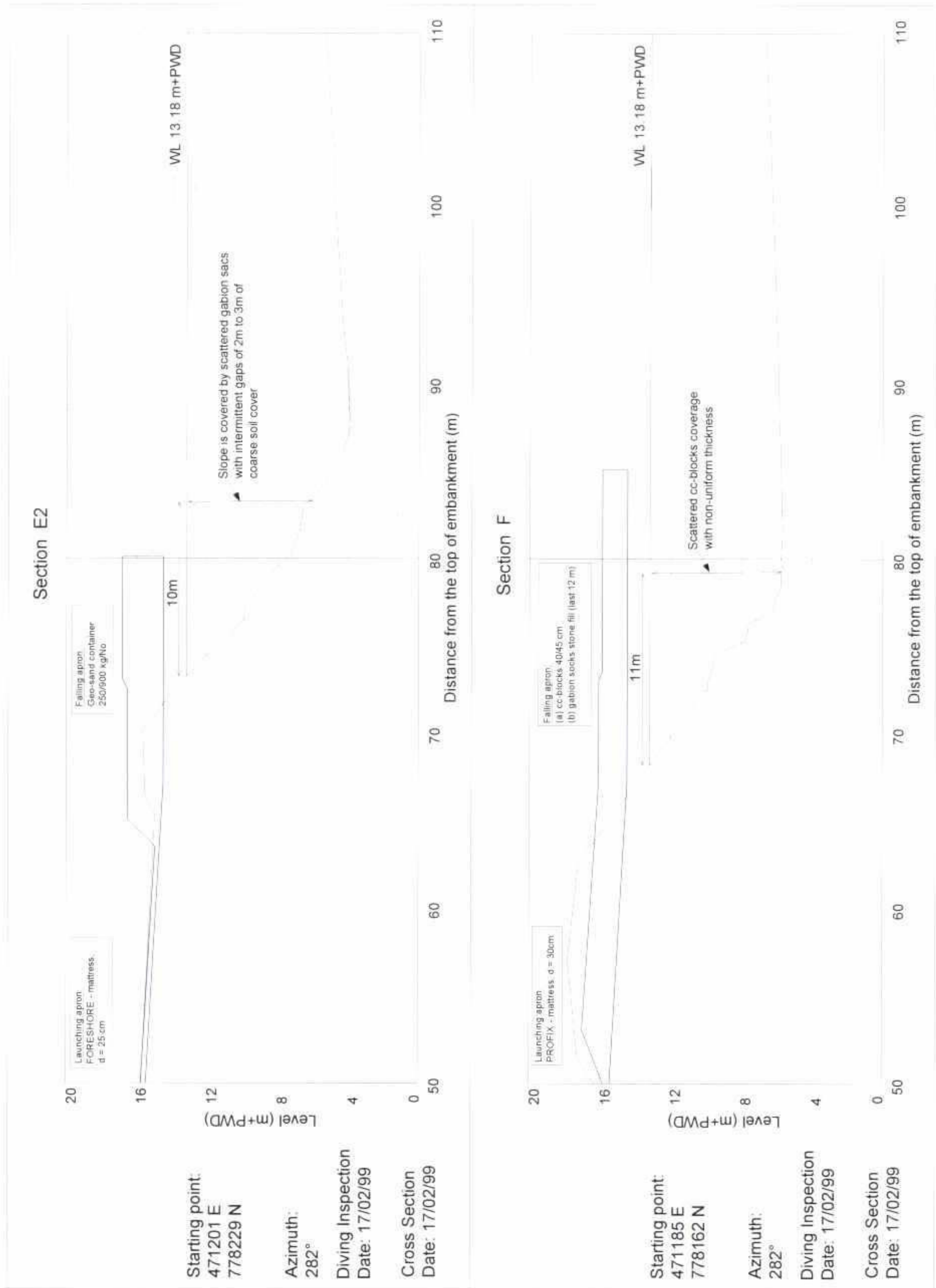


Fig. 5.3-4: Diving inspection at Sections E2 and F

Fig. 5.3-5: Diving inspection at Sections G and H1

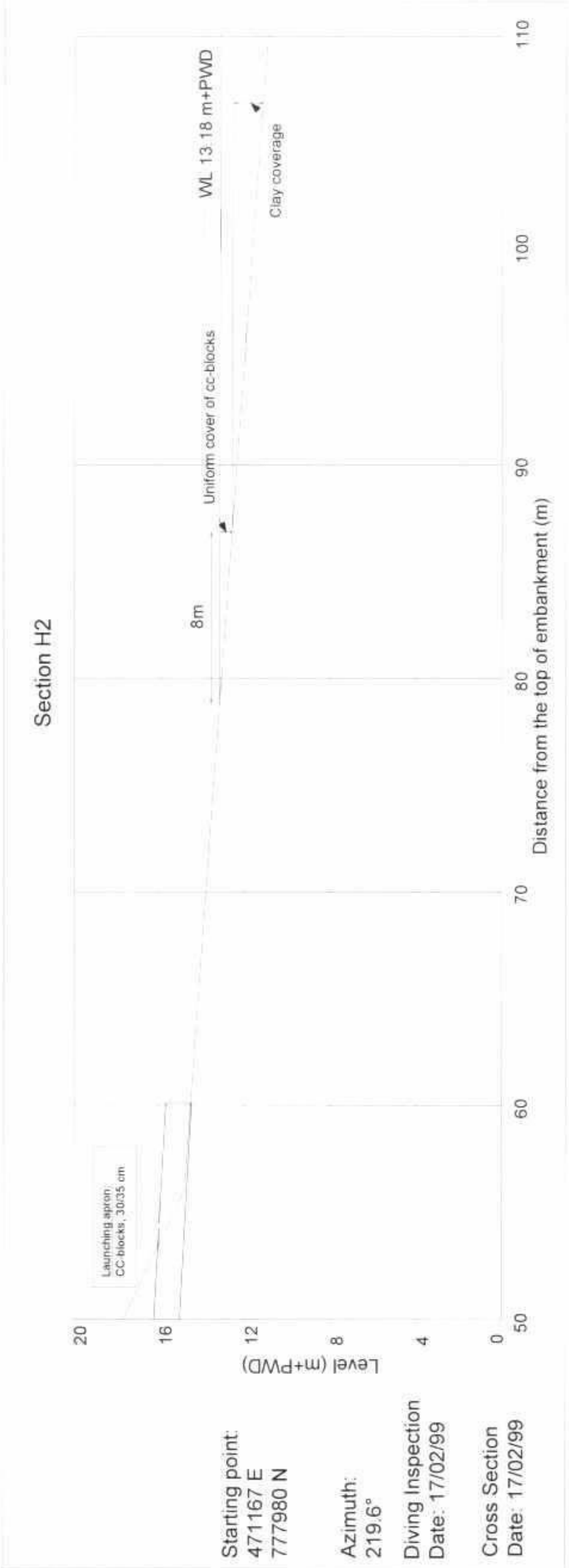


Fig. 5.3-6: Diving inspection at Section H2

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Consulting Consortium FAP 21/22 (Mar. 1999), Monitoring and Adaptation Report at Bahadurabad Test Site, Monsoon 1997

Consulting Consortium FAP 21/22 (Mar. 1999), Monitoring and Adaptation Report at Bahadurabad Test Site, Monsoon 1997, Annexes

Consulting Consortium FAP 21/22 (Dec. 1999), Monitoring and Adaptation Report at Bahadurabad Test Site, Monsoon 1998

River Survey Project FAP 24 (Nov. 1996), Final Report, Annex 1: Surveys

2b

Attachment 1
Logbook Form Sheet

Date: / /	Radio contact with/at [hrs.]		Emergency message reported		
Page 1	Dhaka	CSE	To:	at:	hrs.

METEOROLOGICAL DATA	recorded at:		hrs.		
Temperature [°C]	Rainfall [mm]	Wind Direction	Wind Speed [m/s]		

WATER LEVEL DATA - Gauge readings [m PWD]					
Upstream of the structure			Downstream of the structure		
8.00 hrs.	13.00 hrs.	17.00 hrs.	8.00 hrs.	13.00 hrs.	17.00 hrs.

WAVE OBSERVATIONS					
Upstream gauge			**Downstream gauge**		
Time	Wave height [cm]	Directon	Time	Wave height [cm]	Direction

VISUAL CONTROL OF TEST STRUCTURES (add Special Damage Report, if required)		Time of Inspection: hrs.	
Location	Falling apron	Launching apron	Revetment
A			
B			
C			
D			
E			
F			
G			
H			

BANKLINE OBSERVATIONS (changes, location, progress / use page 2 for sketch, measurements)

MONITORING SURVEYS CARRIED OUT		
Section measurement time:	Time of sending to Dhaka:	
Bathymetry from:	Rolls. No.	
Current Measurements	Location: Cross-section:	Float Trackings:
Bankline Survey:	Topographic Survey:	

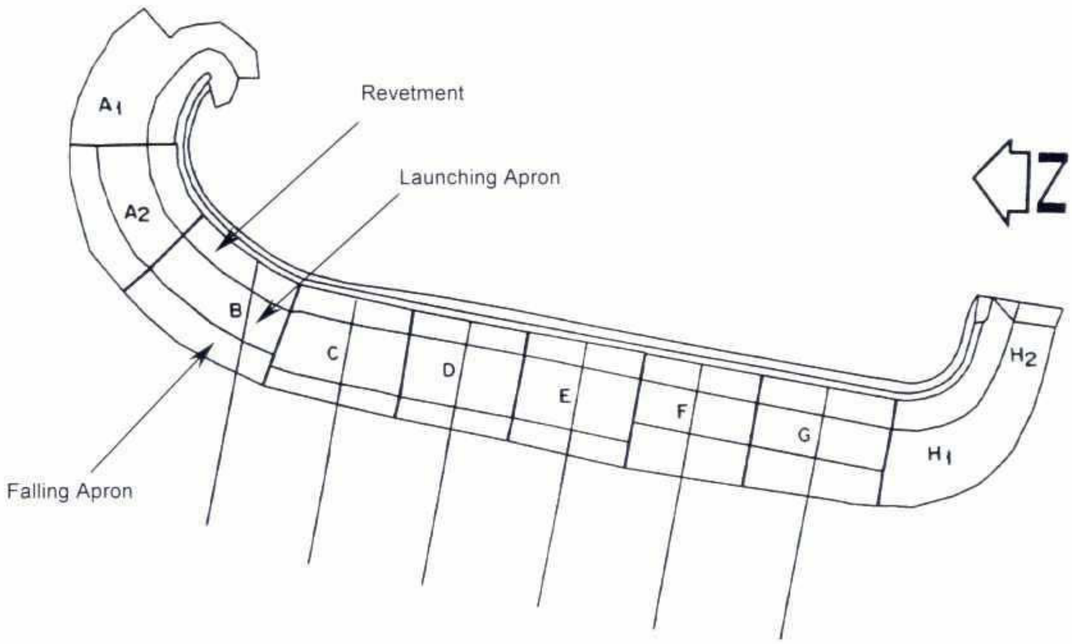
Reported by:	Checked:	Seen CSE:

(to be used for sketches, measurements, special notes, etc. to support the daily observations and activities)	Date: / / Page 2

Photos taken Film No.: Picture Nos.:

Location Plan of Test Structures

(mark location of photos, measurements, special observations, etc.)



Reported by:	Checked:	CSE:

Attachment 2

Hydrographs from

June to September in 1995 to 1999

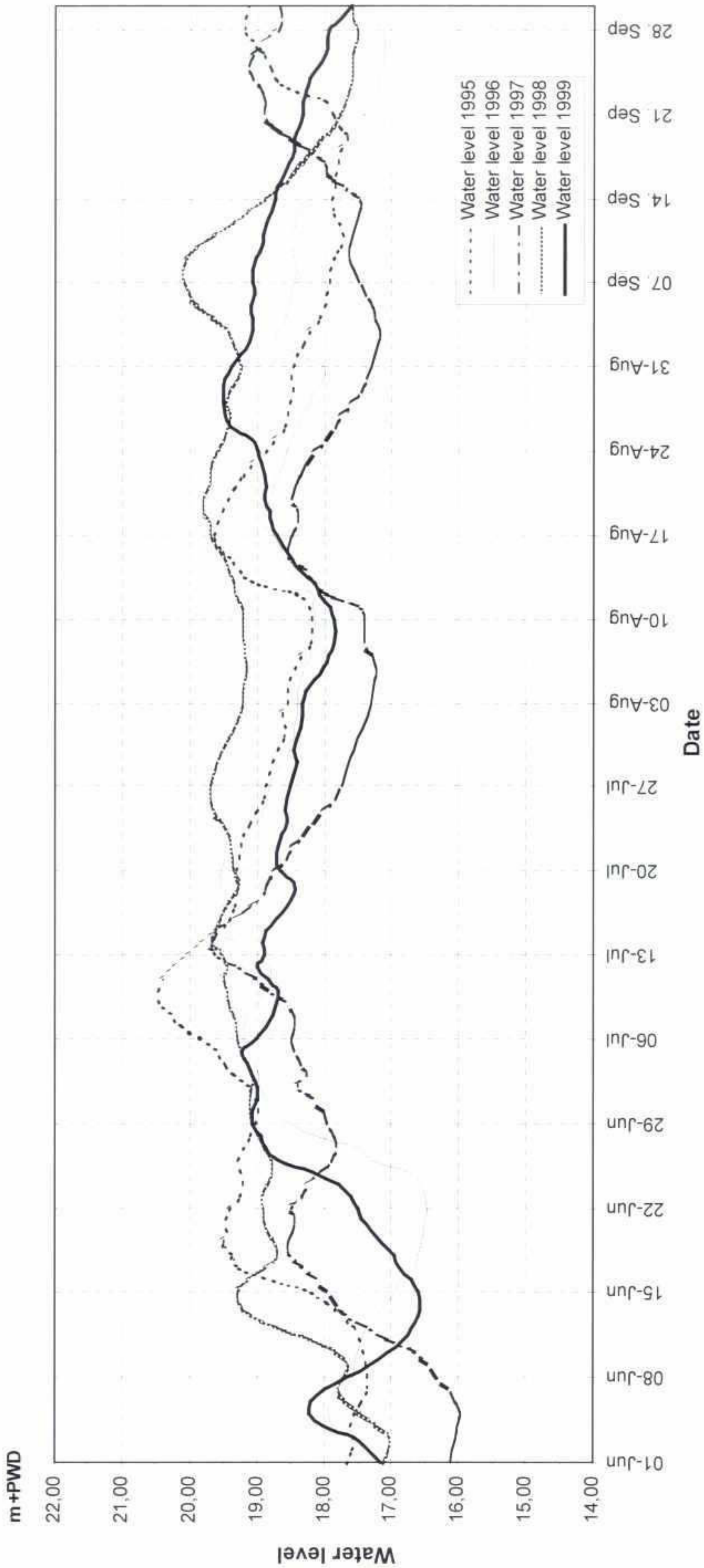


Fig. A.2-1: Hydrographs from June to September in 1995 to 1999

Attachment 3

Precipitation at Bahadurabad

208

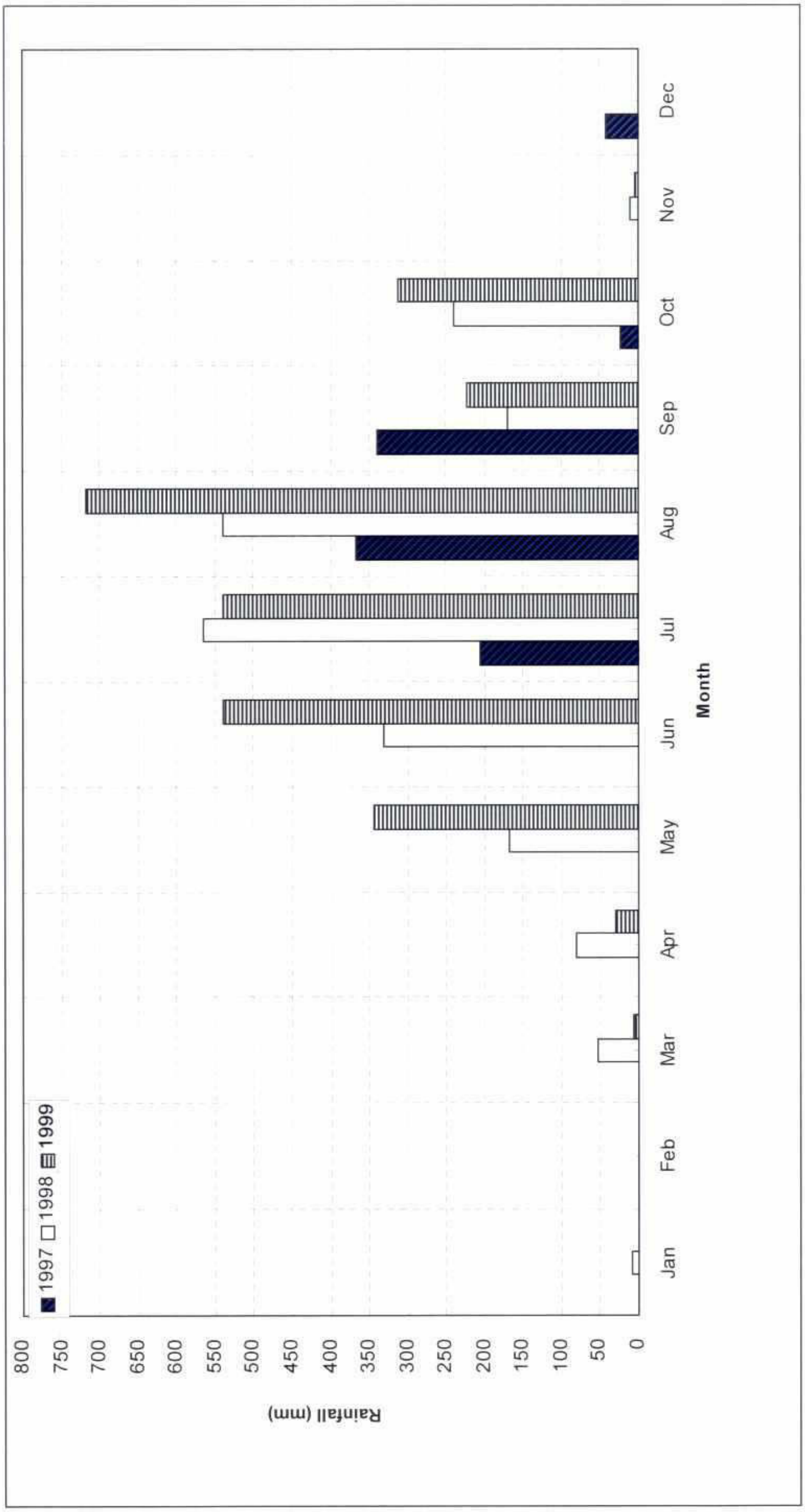


Fig. A.3-1: Precipitation at Bahadurabad

706

Attachment 4

Data Archive System FAP 21

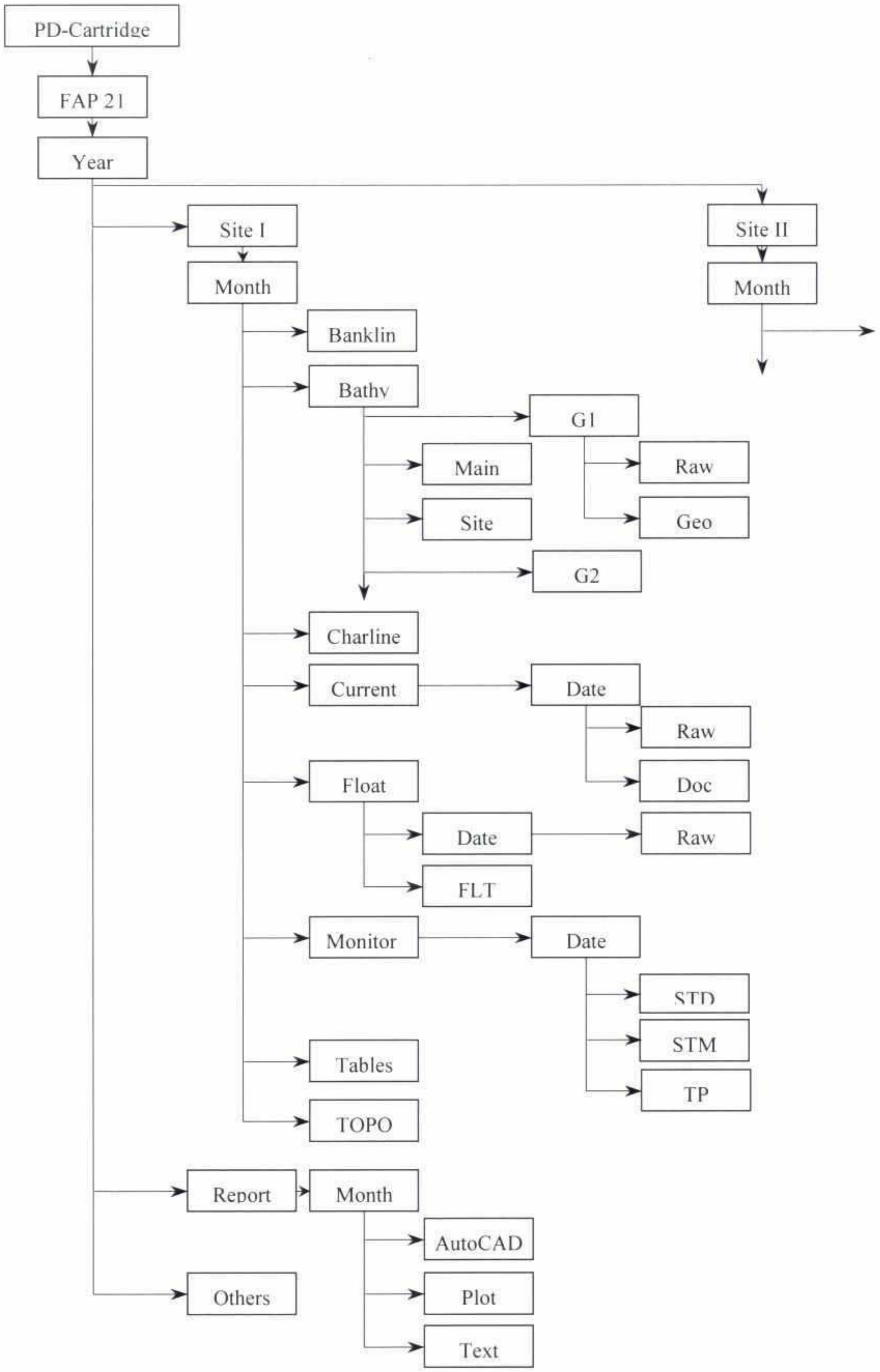


Fig. A.4-1: Data archive system FAP 21

209

Attachment 5

Inventory of Monitoring Data

1 INVENTORY OF MONITORING DATA

1.1 BATHYMETRY

Inventory of Bathymetry Survey
Monsoon 1995 - 1999

Sl. No.	Survey Type*	Test Site-II, Bahadurabad Area (Δ N x Δ E)	Date	Line Spacing (m)	Water Level (m+PWD)
1	M	main channel	11-12/06/95	100	-
2	S	site	14/09/95	100	-
3	S	site	01/11/95	100	-
4	S	site	11/11/95	50	-
5	S	site	16/11/95	50	15.4
6	S	site	04/12/95	50	14.6
7	S	site	11/12/95	50	14.3
8	S	site	18/12/95	50	14.2
9	S	site	15/04/96	50	14.2
10	S	site	27/05/96	50	17.1
11	S	site	11-12/07/96	50	18.8-18.8
12	S	site	30/07/96	50	18.5
13	S	site	21/08/96	50	18.8
14	S	site	02/09/96	50	18.7
15	S	site	15-17/09/96	50	18.2-17.9
16	S	site	09/10/96	20, 60	17.7
17	S	site	09/12/96	20, 40	14.4
18	S	site	02/01/97	40, 20	13.8
19	S	site	04/02/97	40	16.3
20	S	site	08/03/97	40	13.2
21	S	site	04/04/97	40	14.4
22	S	site	06/05/97	20, 40	14.2
23	S	site	11-12/06/97	40	17.4
24	S	site	20/06/97	20	18.5
25	S	site	28-30/06/97	20, 40	17.9-18.0
26	S	site	12/07/97	20	19.3
27	M	main channel (2500m x 1500m)	19-22/7/97	40	18.9-18.5
28	S	site	01/08/97	20	17.5
29	M	main channel (2500m x 1600m)	07-08/08/97	40	17.3-17.4
30	SP	section H to 500m d/s	11/08/97	20	17.5
31	S	site	15/08/97	20	18.5
32	M	main channel (2500m x 1500m)	20-21/08/97	40	18.5
33	S	site	01/09/97	20	17.3
34	M	main channel (2500m x 1500m)	13-14/09/97	50	17.5
35	S	site	25/09/97	20	19.1

Sl. No.	Survey Type*	Test Site-II, Bahadurabad Area (Δ N x Δ E)	Date	Line Spacing (m)	Water Level (m+PWD)
36	M	main channel (2500m x 1500m)	04-06/10/97	50	18.3-18.2
37	M	main channel (2500m x 1500m)	20-21/10/97	50	16.3-16.2
38	M	main channel (2500m x 1500m)	08-09,11/11/97	50	15.0-14.9
39	M	main channel (2500m x 1500m)	17-18/12/97	50	14.0
40	M	main channel (2500m x 1500m)	10-11/01/98	50	13.4
41	M	main channel (2500m x 1500m)	27/01/98	50, 100	13.2
42	S	site	05/02/98	10	11.8
43	M	main channel (2500m x 1500m)	08/02/98	50, 100	13.0
44	M	Ghutail to structure site	17/02/98	300	13.0
45	M	main channel (2500m x 1500m)	01-02/03/98	-	13.1-13.2
46	M	main channel (2500m x 1500m)	18/03/98	50, 100	13.5
47	M	main channel (2500m x 1500m)	04-05/04/98	100	14.7
48	S	site	04/04/98	25	14.7
49	S	around section H	21/04/98	20	14.7
50	S	around section H	26/04/98	20	14.7
51	S	site	27/04/98	25	14.7
52	M	main channel (1200m x 1000m)	28/04/98	100	14.7
53	S	site	08/05/98	25	15.5
54	M	main channel (2500m x 1500m)	08-09/05/98	100	15.5-15.4
55	S	site	26-27/05/98	25	15.8-15.9
56	M	main channel (2500m x 1500m)	27/05/98	100	15.9
57	S	site	06-07/06/98	25	17.7-17.8
58	S	site	14-16/06/98	25	19.3-19.1
59	M	main channel (2500m x 1500m)	16-18/06/98	100	19.1-18.7
60	M	main channel (Ghutail to 3km u/s of railway ghat)	25-27/06/98	100	18.8-18.9
61	S	site	11/07/98	25	19.5
62	M	main channel (8000m x 5000m)	17-19,21/07/98	100	19.5-19.4
63	M	main channel (2800m x 2000m)	28-30/07/98	100	19.7-19.5
64	M	bypass channel	06-08/08/98	100	19.2-19.2
65	S	site	09-10/08/98	25	19.2
66	M	main channel (2800m x 2000m)	31/08-03/09/98	100	19.3-19.5
67	S	site	10/09/98	25	19.9

Sl. No.	Survey Type*	Test Site-II, Bahadurabad Area (Δ N x Δ E)	Date	Line Spacing (m)	Water Level (m+PWD)
68	M	main channel (3500m x 2000m)	14,17-19/09/98	50, 100	18.9-18.0
69	M	Belgacha to 1.5km d/s of Ghutail bazar	10/10/98	50	16.5
70	M	main channel (6000m x 4000m)	13-14,16/10/98	100	16.3-16.1
71	S	site	22/10/98	25	16.8
72	M	Harindhara (u/s of railway ghat)	29/10/98	-	17.3
73	S	site	19/11/98	25	15.0
74	M	main channel (3900m x 1000m)	23-24,26/11/98	100	14.8-14.6
75	S	site	06/12/98	25	14.5
76	S	site	11/12/98	-	14.3
77	S	site	30/12/98	25	13.8
78	M	main channel (2000m x 1000m)	31/12/98	-	13.8
79	S	site	15-16/01/99	25, 50	13.6
80	M	main channel (4000m 1000m)	02-04/01/99	100	13.8
81	M	main channel (4000m x 1000m)	18,21,23/01/99	100	13.5
82	M	main channel (4km u/s of railway ghat to 2.5km d/s of Ghutail)	14-18/02/99	50, 100	13.2
83	S	near structure	14/02/99	50	13.2
84	M	Haridhara to 2.5km d/s of Ghutail bazar	10-15/03/99	50, 100	13.2
85	S	site	10/03/99	25	13.2
86	S	site	15/04/99	25	14.1
87	M	main (Haridhara to 2.5km d/s of Ghutail bazar)	15-18/04/99	100	14.1-13.9
88	S	site	04/05/99	25	15.4
89	M	main channel (Haridhara to 2.5km d/s of Ghutail bazar)	24-25/05/99	100	15.1
90	M	main channel (d/s of Ghutail bazar to Harindhara)	16-17/06/99	100	16.7-16.9
91	S	site	18/06/99	25, 50	17.0
92	S	site (777550N to 1000m u/s)	28/06/99	10	18.9
93	M	main channel extended u/sof Harindhara	09-14/07/99	50, 100	18.7-18.9
94	M	special survey from railway ghat to 6km u/s of Harindhara	20-21/07/99	100	18.7-18.7
95	M	railway ghat to u/s of Harindhara	02/08/99	50	18.3
96	M	near suger can office	04/08/99	100	18.3
97	M	main channel (8000m x 4000m)	11-16/08/99	50, 100	17.9-18.6
98	M	special survey (Ghutail area)	29/08/99	-	19.5
99	M	main channel (772 - 782)	09-15/09/99	100	19.0-18.7
100	M	special survey along Belgacha	19/09/99	-	18.4
101	M	special survey (1km u/s and d/s of railway ghat)	30/09/99	100	17.6
102	M	main channel (7300m x 3000m)	10-13/10/99	-	17.1-17.3

222

Sl. No.	Survey Type*	Test Site-II, Bahadurabad Area (Δ N x Δ E)	Date	Line Spacing (m)	Water Level (m+PWD)
103	M	main channel (7800m x 3000m)	27-29/10/99	-	17.7-17.3
104	M	Ghutail area	13/11/99	20	15.7
105	M	main channel (7800m x 3000m)	27-29/11/99	100	15.0-15.0

* M = Main Channel; S = Test Site; SP = Special Survey; G = Groyne Field

1.2 RIVER BANK AND CHAR SURVEYS

Inventory of Bankline and Charline Survey
Monsoon 1995 – 1999

Sl. No.	Survey Type*	Test Site-II, Bahadurabad Area	Date	Water Level (m+PWD)
1	B	site	16/09/95	-
2	B	site	05-06/10/95	-
3	B	site	01/11/95	-
4	B	site	09/12/95	14.4
5	B	site	09/08/96	18.2
6	B	site (bankline, cross-section)	02/09/96	18.7
7	B	site	14/09/96	18.3
8	B	site	09-13/10/96	17.7
9	B	site	12/12/96	14.3
10	B	site	27/05/97	16.2
11	B	railway ghat to 500m d/s of section H	23/07/97	18.3
12	C	deposition in front of section B	23/07/97	18.3
13	B	railway ghat to 1.2km d/s	15/08/97	15.8
14	B	railway ghat to 1.2km d/s	28/08/97	17.6
15	B	section H to southern part	17/09/97	18.0
16	B	railway ghat to 2km d/s of section H	31/10/97	15.4
17	B	Ghutail bazar to structure site	11/02/98	13.0
18	B	test site to Ghutail	17-18/02/98	13.0
19	C	char	17-18/02/98	13.0
20	B	site	19/02/98	13.0
21	B	site	22/02/98	13.0
22	B	section H to 2.5km d/s	02/03/98	13.2
23	B	structure site	08/03/98	13.0
24	B	upto 74000N	04/04/98	14.7
25	B	Ghutail bazar to section H	28/04/98	14.7
26	B	site	13/05/98	15.7
27	B	bankline	01/06/98	17.1
28	B	Ghutail (776000-773000N)	14/09/98	18.9
29	B	Harindhara to section B	05/11/98	16.2

Sl. No.	Survey Type*	Test Site-II, Bahadurabad Area	Date	Water Level (m+PWD)
30	C	char in front of structure	21/11/98	14.9
31	C	char in front of structure	18/12/98	14.1
32	B	1km u/s to 1km d/s of Ghutail	13/02/99	13.2
33	B	u/s of the structure (~1.5km)	22/02/99	13.2
34	B	Harindhara to structure & Belgacha to d/s of Ghutail bazar	14/03/99	13.1
35	C	char	14/03/99	13.1
36	B	railway ghat to structure	22/04/99	13.8
37	C	charline in front of structure	22/04/99	13.8
38	B	structure to Harindhara	01/05/99	14.3
39	B	Harindhara to 3km d/s of Ghutail bazar	17-18/06/99	16.9 -17.0
40	B	section H to 3km d/s of Ghutail bazar	05-06/07/99	19.2-19.0
41	B	u/s of railway ghat to 2km u/s of Harindhara school	21/07/99	18.7
42	B	Bahadurabad ghat to u/s of Harindhara	02/08/99	18.3
43	B	section H to Belgacha	19/08/99	18.8
44	B	section H to 2.5km d/s of Ghutail bazar	23-24/08/99	18.9-19.0
45	B	structure to 2.5km d/s of Ghutail bazar	06/09/99	19.1
46	B	railway ghat to 1.5km u/s of Harindhara	11/09/99	18.9
47	B	Belgacha	19/09/99	18.4
48	C	char in front of Belgacha school	12/10/99	17.2
49	B	section H to 2.5km d/s of Ghutail bazar	13/10/99	17.3
50	C	in front of railway track	14/10/99	17.4
51	B	section C to 4km u/s of railway track	15/10/99	17.4
52	B	Belgacha to 2.5km d/s of Ghutail	04/11/99	16.3
53	C	char in front of Ghutail bazar	04/11/99	16.3
54	B	1.5km length in front of Ghutail bazar	27/11/99	15.0

* B = Bankline; C = Charline

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1.3 DGPS FLOW TRACKING

Inventory of DGPS Float Tracking
Monsoon 1997 – 1999

Sl. No.	Test Site-II, Bahadurabad Area	Date	No. of Tracks	Water Level (m+PWD)
1	site	02/01/97	3	13.8
2	site	04/02/97	3	13.3
3	site	08/03/97	3	13.2
4	site	04/04/97	3	14.6
5	site	14/05/97	4	14.8
6	site	12/06/97	1	17.4
7	site	18/06/97	3	18.5
8	site	20/06/97	3	18.5
9	site	24/06/97	3	18.3
10	main channel (1200m x 600m)	05/07/97	3	18.4
11	main channel (1200m x 600m)	14/07/97	4	19.6
12	main channel (1200m x 600m)	17/07/97	5	19.0
13	site	06/08/97	2	17.3
14	site	09/08/97	3	17.4
15	site	16/08/97	2	18.6
16	site	17/08/97	1	18.5
17	site	08/09/97	7	17.6
18	site	20/09/97	2	18.8
19	site	26/09/97	3	19.1
20	main channel (2500m x 1500m)	22/10/97	5	16.0
21	main channel (2500m x 1500m)	25/11/97	3	14.5
22	main channel (2500m x 1500m)	18/12/97	3	14.0
23	main channel (2500m x 1500m)	11/01/98	2	13.4
24	main channel (2500m x 1500m)	28/01/98	2	13.2
25	main channel (2500m x 1500m)	09/02/98	2	13.0
26	main channel (2500m x 1500m)	03/03/98	2	13.1
27	main channel (2500m x 1500m)	11/03/98	2	13.0
28	main channel (2500m x 1500m)	19/03/98	2	13.4
29	main channel (2500m x 1500m)	05/04/98	3	14.5
30	main channel (near structure)	29/04/98	1	14.8
31	main channel (2500m x 1500m)	30/04/98	3	14.9
32	main channel (2500m x 1500m)	10/05/98	1	15.4
33	main channel (2500m x 1500m)	14/05/98	5	15.8
34	main channel (2500m x 1500m)	31/05/98	5	17.2
35	site	08/06/98	2	17.7
36	site	06/09/98	5	20.0
37	main channel, bypass channel	22/09/98	2	17.7
38	main channel (6000m x 4000m)	15/10/98	4	16.2
39	site	22/10/98	1	16.8
40	Harindhara (u/s of railway ghat)	29/10/98	2	17.3
41	site	02/11/98	3	16.6

Sl. No.	Test Site-II, Bahadurabad Area	Date	No. of Tracks	Water Level (m+PWD)
42	site	18/11/98	2	15.0
43	Harindhara	26/11/98	1	14.6
44	main channel (6000m x 5300m)	27/11/98	3	14.6
45	site	05/12/98	1	14.5
46	site	06/12/98	1	14.5
47	site	16/01/99	1	13.6
48	main channel (west)	18/02/99	2	13.2
49	Belgacha/site	19/02/99	3	13.2
50	site	09/03/99	1	13.1
51	Ghutail	11/03/99	1	13.2
52	Belgacha channel	12/03/99	1	13.2
53	main channel (1200m x 400m)	14/03/99	2	13.1
54	site	14/03/99	2	13.1
55	site	24/03/99	1	13.4
56	site	07/04/99	1	13.3
57	along structure	12/04/99	1	13.6
58	along structure	20/04/99	3	13.9
59	Belgacha	20/04/99	1	13.9
60	site	03-04/05/99	3	15.1-15.4
61	Harindhara	06/05/99	1	16.1
62	along structure	07/05/99	3	16.4
63	along structure	03/06/99	2	17.5
64	along structure	08/06/99	2	17.7
65	Ghutail	18/06/99	2	17.0
66	Harindhara and main channel	19/06/99	2	17.2
67	along structure	23,26/06/99	2	17.6-18.7
68	d/s of section H	26-27,29/06/99	9	18.7-19.1
69	site	08/07/99	6	18.8
70	Harindhara	13-14/07/99	1	18.9
71	d/s of structure to Ghutail	15/07/99	2	18.9
72	Harindhara	21/07/99	1	18.7
73	Harindhara	02/08/99	2	18.3
74	Ghutail bazar	04/08/99	3	18.3
75	site	17/08/99	2	18.7
76	main channel (8000m x 4000m)	17/08/99	1	18.7
77	main channel (8000m x 4000m)	28/08/99	4	19.5
78	Ghutail	30/08/99	4	19.5
79	Ghutail	09/09/99	3	19.0
80	main channel (8300m x 4000m)	16/09/99	4	18.6
81	main channel (8300m x 4000m)	17/09/99	2	18.5
82	main channel (7300m x 3000m)	14/10/99	5	17.4
83	main channel (7300m x 3000m)	15/10/99	2	17.4
84	main channel (7300m x 3000m)	30/10/99	3	17.1
85	Ghutail	13-14/11/99	5	15.7
86	main channel (7300m x 3000m)	30/11/99	6	14.9

1.4 ADVANCED DGPS FLOW TRACKING

Inventory of Advanced DGPS Float Tracking
Monsoon 1996 - 1998

Sl. No.	Test Site-II, Bahadurabad Area	Date	Drogue Depth ()	No. of Tracks	Water Level (m+PWD)
1	site	09/08/96	0, 3	6+8	18.2
2	main channel	10/08/96	0, 1.5, 3, 6	14	18.2
3	site	14/08/96	0, 3	13+11	18.5
4	main channel	16/08/96	0, 1.5, 3	12+12+12	18.2
5	site	20/08/96	1.5, 3	7+7	18.8
6	site	23/08/96	1.5	10	18.6
7	main channel	13/06/97	0, 3, 6	6+7+5	17.7
8	main channel	19/06/97	0, 3, 6	9+7+7	18.6
9	main channel	29/06/97	3	11	18.0
10	main channel	02/07/97	0, 3, 6	12+11+11	18.4
11	main channel	08/07/97	0, 3, 6	9+9+9	18.4
12	main channel	13/07/97	0, 3, 6	10+6+6	19.6
13	main channel	20-21/07/97	0, 3, 6	7+11+8	18.7-18.6
14	main channel	01/08/97	0, 3, 6	10+7+8	17.5
15	main channel	06/08/97	0, 3, 6	8+9+9	17.3
16	main channel	12/08/97	0, 3, 6	7+8+7	18.0
17	main channel	18/08/97	0, 3, 6	10+10+11	18.40
18	site test site	14-15/06/98	0, 3, 6	16+15+12	19.3
19	main channel	24-28/06/98	0, 1.5, 3	33+35+31	18.9
20	site test site	09-10/07/98	0, 3, 6	17+17+12	19.4-19.5
21	main channel	10-14/07/98	0, 1.5, 3	31+30+29	19.5
22	Ghutail	28-29/07/98	0, 1.5, 3	20+20+20	19.7-19.6
23	site	29-30/07/98	0, 1.5, 3	18+19+16	19.6-19.5
24	main channel	17-18,20-24, 26-27/08/98	0, 1.5, 3	34+36+29	19.7

1.5 CURRENT POINT MEASUREMENTS

Inventory of Valeport Measurement
Monsoon 1996 - 1999

Sl. No.	Test Site-II, Bahadurabad Area	Date	No. of Verticals	Water Level (m+PWD)
1	site	24/02/97	3	13.2
2	main channel	14/05/97	6	14.8
3	site (along the structure)	11/09/97	7	17.6
4	site (section B to H)	31/03-01/04/98	22	14.6
5	site (section B to H)	29-30-04/98	18	14.8 -14.9
6	site (section B to H)	09-10/05/98	21	15.4
7	site (section B to H)	29-30/05/98	21	17.1 -17.4
8	site (section B to H)	07-08/06/98	21	17.8 -17.7
9	site (section E and F)	22/06/98	4	18.9
10	site (section B, E, G and H)	04/08/98	13	19.2
11	main channel (bypass channel)	05/08/98	11	19.2
12	site (section B, E1and G)	15/09/98	9	18.7
13	site (section B, D, F and H1)	31/10/98	11	16.9
14	site (section B, C, D, F and H) and d/s of bandal structure (Ba/2, Ba/3)	20-21/11/98	21	14.9
15	site (section C, E1 and G)	07/01/99	9	13.7
16	site (section B, C, D, E2 and G)	31/01/99	13	13.4
17	site (section B, C, D, E2, F and H1)	04/03/99	17	13.1
18	Ghutail	06/03/99	5	13.1
19	site (section B, C and D)	05/06/99	7	18.2
20	site (section E1, F and G)	09/06/99	8	17.3
21	site (around d/s termination)	28-29/06/99	16	18.9-19.1
22	site and Ghutail	09/09/99	15	19.0
23	in front of structure	30/09/99	9	17.6

1.6 DISCHARGE MEASUREMENTS

Inventory of Discharge Measurement

Monsoon 1998

Sl. No.	Test Site-II, Bahadurabad Location/Cross-section	Date	Area (m2)	Q (m3/s)	Water Level (m+PWD)
1	main channel (d/s of site)	05/08/98	16332	26826	21.5
2	site	05/08/98	5726	9169	21.5
3	site	20/11/98	1424	1279	16.8
4	site (d/s of bandal structure)	20/11/98	827	708	16.8

1.7 CROSS SECTIONS

Inventory of Cross Sections
Monsoon 1997 - 1999

Sl. No.	Survey Type*	Month	Test Site-II, Bahadurabad Cross Sections									Total Number
			B	C	D	E1	E2	F	G	H1	H2	
1	CS	Jun. '97	4	5	5	5		5	5	1		30
2	CS	Jul. '97	12	24	24	24		24	23			131
3	CS	Aug. '97	18	17	17	20	3	20	20	6		121
4	CS	Sep. '97	11	12	12	15	12	21	21	21		125
5	CS	Oct. '97	5	5	6	6	6	6	6	6	5	51
6	CS	Nov. '97	6	6	6	6	4	7	7	5	6	53
7	CS	Dec. '97	3	3	3	3	1	3	3	3	3	25
8	CS	Jan. '98	4	4	4	4	4	4	4	4	4	36
9	CS	Feb. '98	4	4	4	4	4	4	4	4	4	36
10	CS	Mar. '98	3	3	3	3	3	3	3	3	3	27
11	CS	Apr. '98	5	5	5	5	4	5	5	5	4	43
12	CS	May '98	6	6	5	5	5	5	5	5	5	47
13	CS	Jun. '98	6	6	6	6	6	6	6	6	6	54
14	CS	Jul. '98	6	6	6	6	6	6	6	6	6	54
15	CS	Aug. '98	1	1	1	1	1	1	1	1	1	9
16	CS	Sep. '98	7	7	7	7	7	7	7	7	7	63
17	CS	Oct. '98	4	4	4	4	3	4	3	3	4	33
18	CS	Nov. '98	3	3	3	3	1	3	3	3	3	25
19	CS	Dec. '98	4	4	4	4	4	4	4	4	4	36
20	CS	Jan. '99	3	3	3	3	3	3	3	3	3	27
21	CS	Feb. '99	1	1	1	1	1	1	1	1	1	9
22	CS	Mar. '99	4	4	4	4	4	4	4	3	1	32
23	CS	Apr. '99	3	3	3	3	3	3	3	3		24
24	CS	May '99	2	2	2	2	2	2	2	2		16
25	CS	Jun. '99	6	7	6	5	6	8	7	8	8	61
26	CS	Jul. '99	8	8	6	7	6	8	7	7	6	63
27	CS	Aug. '99	2	2	2	2	3	3	3	3	3	23
28	CS	Sep. '99	3	3	3	3	3	3	3	3	3	27
29	CS	Oct. '99	2	2	2	2	2	2	2	2		16

* CS = Cross Sections

BENCHMARK DESCRIPTION
FAP 21

Point No.	**2018**
Location	Kulkandi
BM – Type	CC-Block (30cm x 30cm x 30cm)
Height above surface	(-) 0.05m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	06/04/97
Access	By car or motor bike
Remarks	No connecting points

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2018			

Coordinates

	Everest Modified	WGS '84
Easting (BTM), [m]	471819.34	
Northing (BTM), [m]	777771.66	
Height (PWD), [m]	19.16	
Latitude [°,','''''']	25°7.05306′ N	25°7.08513′ N
Longitude [°,','''''']	89°43.22578′ E	89°43.05658′ E

Sketch:

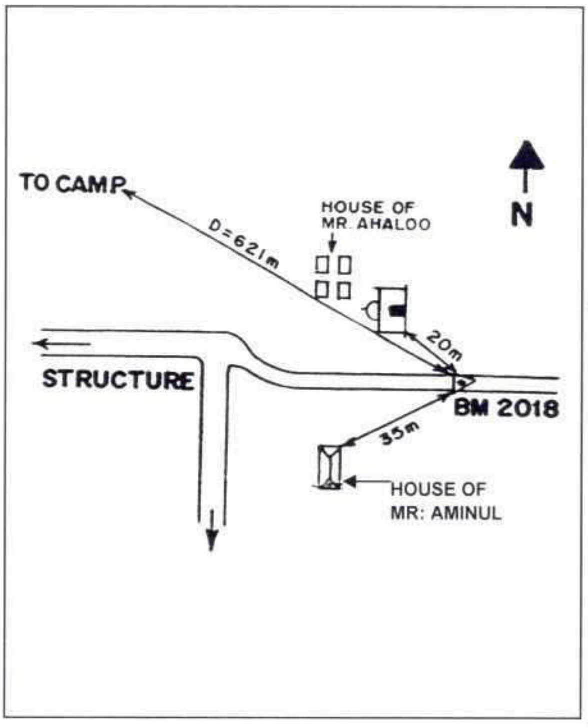


Photo: 193/9 **Date: 08/04/97**

222

BENCHMARK DESCRIPTION
FAP 21

Point No.	**2020**
Location	Kulkandi
BM – Type	CC-Block (30cm x 30 cm x 30 cm)
Height above surface	0.0m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	06/04/97
Access	By Motor bike
Remarks	

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2020	2021	229.293	156.06

Coordinates

	Everest Modified	WGS '84
Easting (BTM), [m]	471602.96	
Northing (BTM), [m]	777325.09	
Height (PWD), [m]	19.03	
Latitude [°,','''''']	25°6.81083′ N	25°6.84291′ N
Longitude [°,','''''']	89°43.09755′ E	89°42.92836′ E

Sketch:

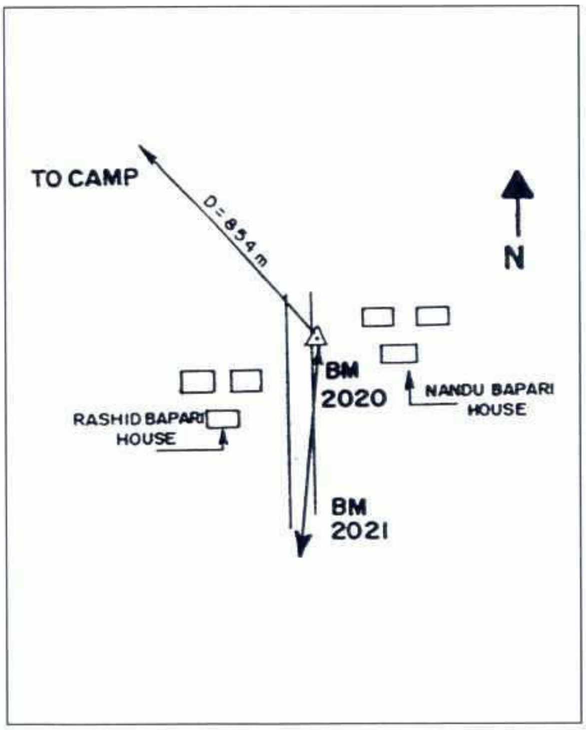


Photo: 193/13 **Date: 08/04/97**

BENCHMARK DESCRIPTION
FAP 21

Point No.	**2021**
Location	Kulkandi
BM – Type	T. Size R.C.C Pillar
Height above surface	0.2m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	1993 (Resurvey: 08/04/97)
Access	By motor bike
Remarks	

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2021	2020	29.293	156.06

Coordinates

	Everest Modified	WGS '84
Easting (BTM), [m]	471533.66	
Northing (BTM), [m]	777185.26	
Height (PWD), [m]	19.92	
Latitude [°,'.'''''']	25°6.73498′ N	25°6.76706′ N
Longitude [°,'.'''''']	89°43.05648′ E	89°42.88730′ E

Sketch:

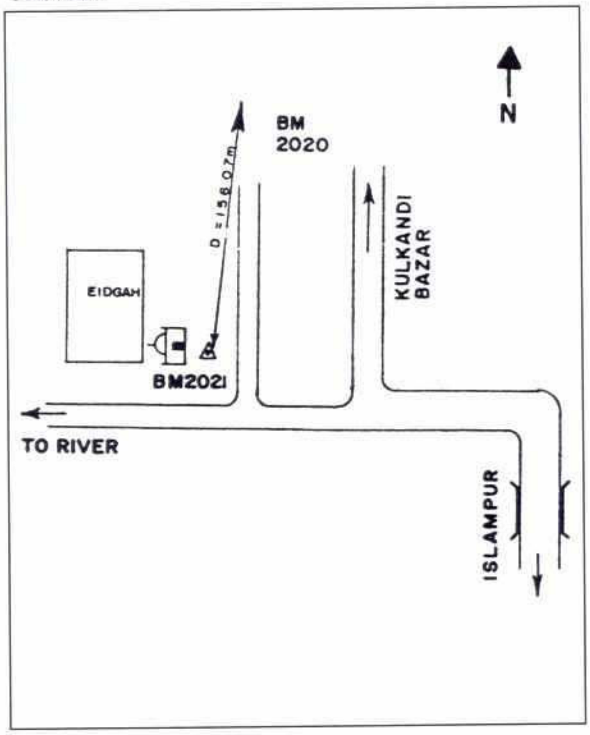


Photo: 193/11 **Date: 08/04/97**

BENCHMARK DESCRIPTION
FAP 21

Point No.	**2024**
Location	Kulkandi
BM – Type	T. Size R.C.C Pillar
Height above surface	0.2m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	1993 (Resurvey: 08/04/97)
Access	By car or motor bike
Remarks	

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2024	2025	284.100	118.62

Coordinates

	Everest Modified	WGS '84
Easting (BTM), [m]	471325.118	471037.089
Northing (BTM), [m]	777806.605	778088.016
Height (PWD), [m]	20.14	
Latitude [°,'.'''''']	25°7.07145′ N	25°7.10350′ N
Longitude [°,'.'''''']	89°42.93156′ E	89°42.76240′ E

Sketch:

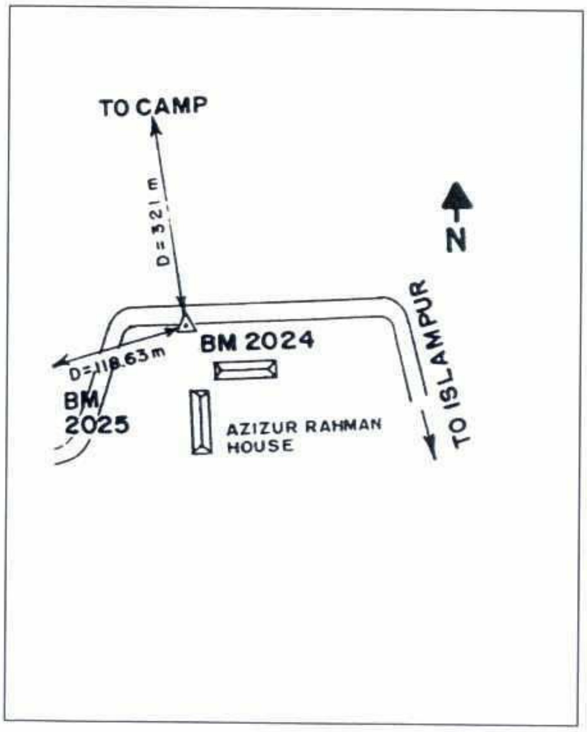


Photo: 193/2 **Date: 08/04/97**

BENCHMARK DESCRIPTION
FAP 21

Point No.	**2025**
Location	Kulkandi
BM – Type	T. Size R.C.C Pillar
Height above surface	0.1m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	06/04/97
Access	By motor bike
Remarks	

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2025	2024	84.100	118.62

Coordinates

	Everest Modified	WGS '84
Easting (BTM) , [m]	471210.18	
Northing (BTM) , [m]	777777.28	
Height (PWD) , [m]	19.24	
Latitude [°,'.'''''']	25°7.05542′ N	25°7.08748′ N
Longitude [°,'.'''''']	89°42.86320′ E	89°42.69403′ E

Sketch:

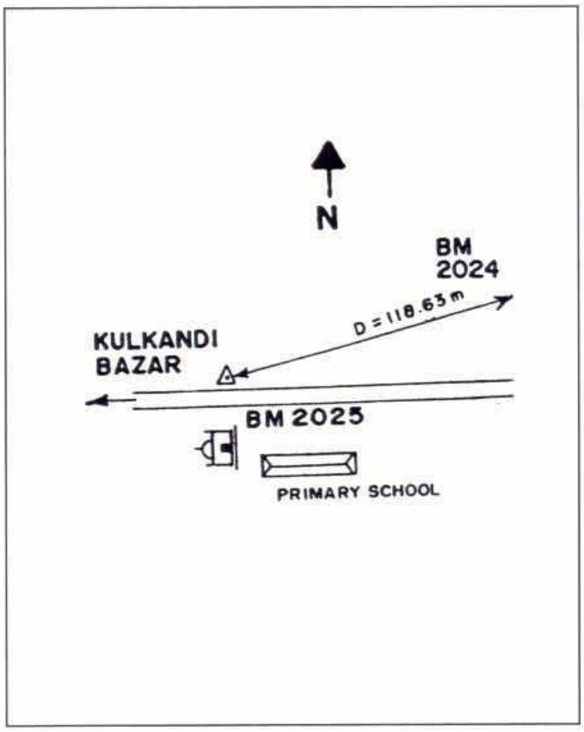
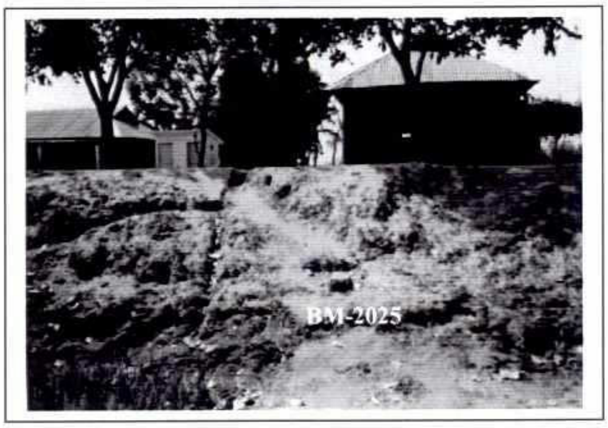


Photo: 193/15 **Date: 08/04/97**



BENCHMARK DESCRIPTION
FAP 21

Point No.	**2026**
Location	Bahadurabad FAP Camp
BM – Type	T. Size R.C.C. Pillar
Height above surface	0.10m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	1996
Access	By car or motor bike
Remarks	

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2026	2024	197.0538	320.76
	2027	327.5725	41.89

Coordinates

	Everest Modified	WGS '84
Easting (BTM), [m]	471310.28	
Northing (BTM), [m]	778127.02	
Height (PWD), [m]	20.90	
Latitude [°,'.'''''']	25°7.24507′ N	25°7′.27710′ N
Longitude [°,'.'''''']	89°42.92233′ E	89°42.75317′ E

Sketch:

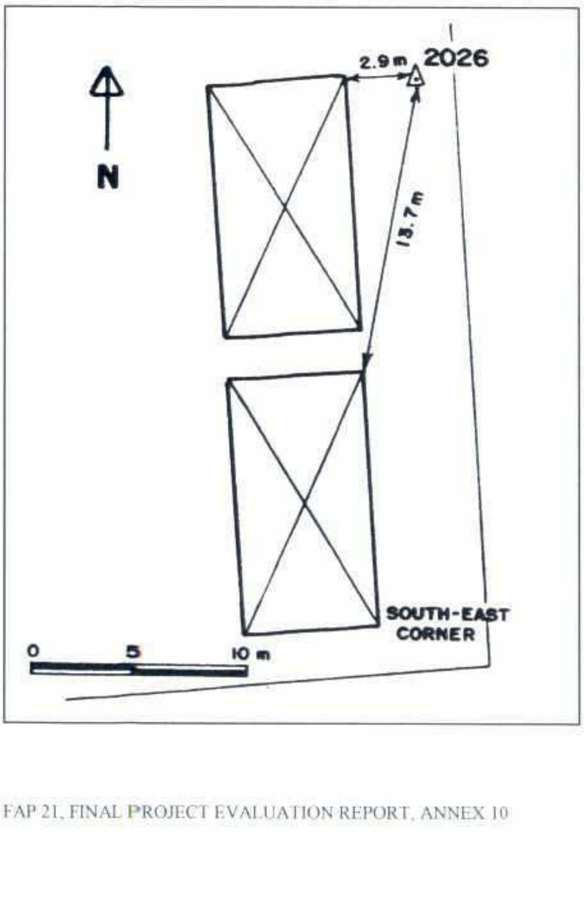


Photo: 358/4 **Date: 04/02/2000**

BENCHMARK DESCRIPTION
FAP 21

Point No.	**2027**
Location	Bahadurabad FAP Camp
BM – Type	Cast-in-Situ R.C.C. Pillar
Height above surface	0.45m
Instrument	EDM, Wild TC 1600
Surveyor	Osman Ghani
1st Installation	1996 (Resurvey: 08/04/97)
Access	By car or motor bike
Remarks	

Connecting Fixpoints

Location Point No.	Target Point No.	Horizontal Angle [gon]	Distance [m]
2027	2026	127.5725	41.89
	2032	313.7579	86.23

Coordinates

	Everest Modified	WGS '84
Easting (BTM) , [m]	471272.26	
Northing (BTM) , [m]	778144.60	
Height (PWD) , [m]	21.355	
Latitude [°,'.'''''']	25°7.25455′ N	25°7.28658′ N
Longitude [°,'.'''''']	89°42.89968′ E	89°42.73052′ E

Sketch:

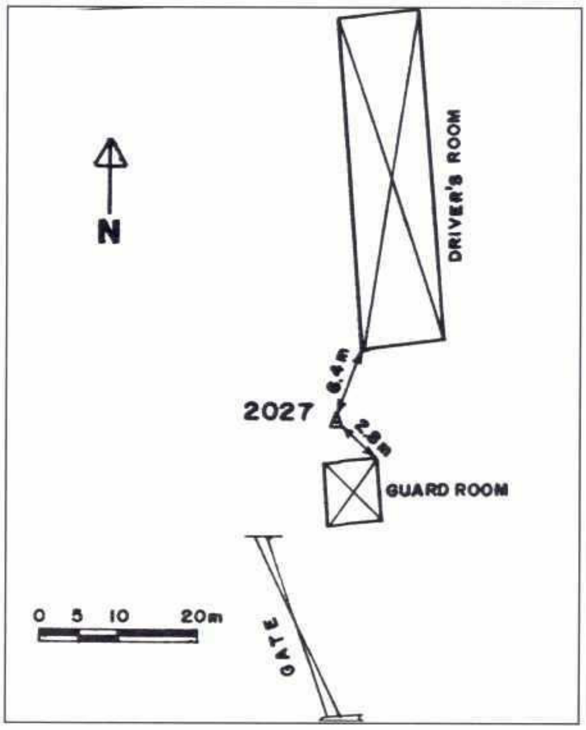


Photo: 358/3 **Date: 04/02/2000**

126

BANK PROTECTION PILOT PROJECT
FAP 21

FINAL PROJECT EVALUATION REPORT

ANNEX 11

THE REVETMENT TEST STRUCTURE EVALUATION OF HYDRAULIC LOADS AND RIVER RESPONSE

AUGUST 2001

22

FAP 21 - BANK PROTECTION PILOT PROJECT

FINAL PROJECT EVALUATION REPORT

ANNEX 11

Table of Contents

				Page
List of Acronyms				**A-1**
Glossary				**G-1**
List of Symbols				**S-1**
SUMMARY				**0-1**
1	**INTRODUCTION**			**1-1**
2	**APPROACH FOR ESTIMATION OF HYDRAULIC AND LOCAL MORPHOLOGICAL PARAMETERS**			**2-1**
3	**BASIC DESIGN CONCEPT**			**3-1**
4	**EVALUATION OF HYDRAULIC LOADS FOR THE DESIGN**			**4-1**
	4.1	INTRODUCTION		4-1
	4.2	WATER LEVELS		4-1
		4.2.1	Monitoring	4-1
		4.2.2	Peak Flows	4-2
		4.2.3	Rate of Rise and Fall	4-4
		4.2.4	Water Level Gradient	4-6
	4.3	FLOW VELOCITIES AND FLOW PATTERN		4-9
		4.3.1	Valeport Measurements	4-9
		4.3.2	Float Tracking	4-12
		4.3.3	Flow Pattern	4-16
		4.3.4	Comparison with Physical Model Tests and Design Flow	4-20
		4.3.5	Estimation of Roughness Coefficients	4-20
	4.4	WIND AND WAVES		4-21
		4.4.1	Wind Speed and Wind Direction	4-21
		4.4.2	Waves	4-22
5	**EVALUATION OF STRUCTURAL BEHAVIOUR**			**5-1**
	5.1	COVER LAYERS AND FILTER LAYERS		5-1
		5.1.1	Damages	5-1
		5.1.2	Physical Explanation	5-1
	5.2	LAUNCHING APRON		5-3

			Page
	5.3	FALLING APRON	5-4
		5.3.1 Monitoring	5-4
		5.3.2 Slopes	5-6
		5.3.3 Erosion Rates	5-10
		5.3.4 Coverage of Falling Apron	5-11
6	**RIVER RESPONSE**		**6-1**
	6.1	CHANGES UPSTREAM AND DOWNSTREAM FROM THE TEST STRUCTURE	6-1
	6.2	CHANGES IN THE TEST SITE AREA	6-2
7	**SCOURING**		**7-1**
	7.1	INTRODUCTION	7-1
	7.2	LOCATION OF THE MAXIMUM SCOUR DEPTH	7-1
	7.3	MAXIMUM SCOUR DEPTHS	7-2
		7.3.1 Definitions	7-2
		7.3.2 Observed Scour Depths and Erosion Rates	7-2
	7.4	SCOUR SLOPES AND VOLUME	7-12
		7.4.1 Slope in Longitudinal Direction	7-12
		7.4.2 Slopes in Transversal Direction	7-14
		7.4.3 Volume of Scour Holes	7-15
	7.5	RISK OF SLIDES	7-15
	7.6	FORMATION OF CONFLUENCE SCOUR	7-15
	7.7	COMPARISON WITH PHYSICAL MODEL INVESTIGATIONS	7-16
8	**RECOMMENDATIONS FOR FUTURE DESIGN ASSUMPTIONS**		**8-1**
	8.1	HYDRAULIC DESIGN PARAMETERS	8-1
	8.2	MORPHOLOGICAL DESIGN PARAMETERS	8-1
		8.2.1 Design Scour Depth	8-1
		8.2.2 Equilibrium Structure Induced Scour Depth	8-2
		8.2.3 Time Dependent Development of the Scour Hole	8-3
REFERENCES			**R-1**

Page

LIST OF TABLES

Table 4.2-1:	Maximum water levels in 1997 and 1998 (the given return periods are approximate)	4-3
Table 4.3-1:	Depth averaged flow velocities (m/s) and standard deviations (m/s) above the face of the falling apron 1998	4-11
Table 4.3-2:	Maximum flow velocities measured above the launching aprons by float tracking in 1997	4-13
Table 4.3-3:	Maximum flow velocities measured above the falling apron by float tracking on July 13, 1997	4-14
Table 4.3-4:	Empirical roughness of sloping face and adjacent riverbed from measurements in 1998	4-21
Table 5.1-1:	Critical gradients from filter test investigations	5-3
Table 5.3-1:	Observation of the development of the falling aprons during the 1997 flood	5-4
Table 5.3-2:	Two small slides in falling apron process in 1997	5-8
Table 5.3-3:	Soil parameters in Bahadurabad	5-10
Table 5.3-4:	Average erosion/sedimentation rates [m/day] in Section F in 1997 monsoon	5-11
Table 7.3-1:	Maximum scour depth near revetment in 1998	7-8
Table 7.4-1:	Longitudinal slopes and volume of the scour hole measured 1997	7-14
Table 7.4-2:	Side slopes of the scour hole measured in 1997	7-15
Table 8.4-1:	Coefficient for scour depth from measurements in 1997	8-6

LIST OF FIGURES

Fig. 4.2-1:	Water levels measured upstream from the railway ghat and at the test structure 1998	4-2
Fig. 4.2-2:	Hydrographs in 1997 and 1998 in a statistical frame	4-3
Fig. 4.2-3:	Duration curves of non-exceeded water levels for 1997 and 1998	4-4
Fig. 4.2-4:	Daily rise and fall of water level at upstream gauge 1998	4-5
Fig. 4.2-5:	Daily rise and fall of water level at the downstream gauge 1998	4-5
Fig. 4.2-6:	Rise and fall of water level in [m/hour] at upstream gauge in summer 1998	4-6
Fig. 4.2-7:	Water level gradient in 1998	4-7
Fig. 4.2-8:	Water level gradient as function of water level rise and fall 1998	4-7
Fig. 4.2-9:	Water level gradients upstream and in front of test structure from May until July 1998	4-8
Fig. 4.2-10:	Water level gradient as function of the water level in 1997	4-9
Fig. 4.3-1:	Locations and numbering of flow velocity verticals	4-10
Fig. 4.3-2:	Position for flow velocity measurement	4-10
Fig. 4.3-3:	Depth averaged flow velocities on September 11, 1997	4-11
Fig. 4.3-4:	Depth averaged flow velocities with standard deviation along the test structure on May 29, 1998	4-12
Fig. 4.3-5:	Example of float tracking on August 12, 1997	4-13

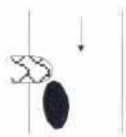
		Page
Fig. 4.3-6:	Measured flow velocities at different cross-sections and design velocity profile	4-15
Fig. 4.3-7:	Calculated depth averaged flow velocities from measurements and design design velocity distribution along the revetment slope	4-15
Fig. 4.3-8:	Flow pattern and bathymetry during June 07 to 16, 1998	4-17
Fig. 4.3-9:	Flow pattern and bathymetry during the period June 22 to 28, 1998	4-18
Fig. 4.3-10:	Flow pattern and bathymetry during the period July 10 to August 04, 1998	4-19
Fig. 4.4-1:	Frequency of wind speed and direction in 1997 and 1998	4-22
Fig. 4.4-2:	Frequency of wave height in 1997 and 1998	4-23
Fig. 5.1-1:	Rain cuts under layer of concrete slabs in Section E	5-2
Fig. 5.3-1:	Steep slopes in front of the falling apron in Section H	5-7
Fig. 5.3-2:	Cross-section H-1 with falling apron	5-9
Fig. 5.3-3:	Cross-section H-1, on August 26, 1997	5-12
Fig. 5.3-4:	Action at a river bend according to Spring (1903)	5-13
Fig. 6.1-1:	Bank erosion from June 1997 to June 1998	6-1
Fig. 6.1-2:	Impossibility to assess net effect of impeded bank erosion at test site and embayment formation immediately downstream	6-2
Fig. 7.2-1:	Location of the deepest point in the scour hole 1997	7-2
Fig. 7.3-1:	Development of bed level and scour depth in 1997	7-3
Fig. 7.3-2:	Bathymetry on January 10/11, 1998	7-4
Fig. 7.3-3:	Bathymetry on February 08, 1998	7-4
Fig. 7.3-4:	Bathymetry on March 18, 1998	7-5
Fig. 7.3-5:	Bathymetry on April 04/05, 1998	7-5
Fig. 7.3-6:	Bathymetry on May 26/27, 1998	7-6
Fig. 7.3-7:	Bathymetry on June 06/07, 1998	7-6
Fig. 7.3-8:	Bathymetry on June 14 to 18, 1998	7-7
Fig. 7.3-9:	Bathymetry on June 25 to 28, 1998	7-7
Fig. 7.3-10:	Bathymetry on July 17 to 30, 1998	7-8
Fig. 7.3-11:	Bathymetry on September 10, 1998 with two section lines	7-9
Fig. 7.3-12:	Dunes and bedforms measured September 10, 1998	7-9
Fig. 7.3-13:	Bathymetry on September 14 to 19, 1998	7-10
Fig. 7.3-14:	Bathymetry on October 13 to 16, 1998	7-10
Fig. 7.3-15:	Bathymetry on November 23 to 26, 1998	7-11
Fig. 7.3-16:	Bathymetry on December 31, 1998	7-11
Fig. 7.4-1:	Slopes in longitudinal sections of the scour hole	7-12
Fig. 7.4-2:	Bathymetry in August 1997 with longitudinal section A-A'	7-13
Fig. 7.4-3:	Bathymetry in September 1997 with longitudinal section B-B'	7-13
Fig. 7.4-4:	Bathymetry in October 1997 with longitudinal section C-C'	7-16
Fig. 8.3-1:	Design scour pattern and bank line	8-2

260

LIST OF ACRONYMS

FAP	-	Flood Action Plan
PWD	-	Public Works Department (datum level)
RRI	-	River Research Institute
SHW	-	Standard High Water
SLW	-	Standard Low Water
USWES	-	US Water Experiment Station

GLOSSARY

bend scour	scour in outer bend	
Chézy coefficient	coefficient for hydraulic roughness	
confluence scour	scour occurring where two channels or rivers join	
falling apron	suitably designed layer of granular material, such as concrete blocks or boulders, placed directly on the existing subsoil or river bed (i.e. without filter)	
hydrograph	graph representing discharges or water levels at a given station as a function of time	
launching apron	suitably designed integrated and articulating mattress system, such as sand-filled geosynthetics, concrete-filled geosynthetics or concrete blocks linked to a strong geotextile, placed on prepared slopes above and below water or in a horizontal excavation well above SLW. A launching apron contains a filter	
local scour	scour downstream of a local structure or obstruction	
protrusion scour	scour immediately upstream of a local structure or obstruction	
revetment	protection layer, suitably designed to withstand the forces of the river	
scour	deepening of the bed by erosive action of water	
scour depth	elevation difference between the deepest point of the scour hole and the upstream bed level of the approach channel	
upstream bed level	average level of the riverbed points of a cross-sectional area, which is about 30 to 40% of the total channel cross-sectional area	

D62

LIST OF SYMBOLS

A_l	-	longitudinal sectional area of scour hole	(m^2)
A_t	-	transverse sectional area of scour hole	(m^2)
B	-	flume width / channel width	(m)
b	-	protrusion length of revetment	(m)
C	-	Chézy coefficient	($m^{0.5}/s$)
c	-	cohesion	(kN/m^2)
D	-	block size or stone size	(m)
h	-	water depth	(m)
h_1	-	water depth in approach flow upstream from scour hole	(m)
I	-	water level gradient	(-)
K	-	empirical coefficient	($m^{-1/3}s^{2/3}$)
k_s	-	hydraulic roughness	(m)
n_t	-	exponent in empirical formula for time-dependent scour development	(-)
t	-	time	(s)
t_1	-	characteristic time at which $y_s = h_1$	(s)
U	-	flow velocity	(m/s)
$\overline{u}$	-	depth averaged flow velocity	(m/s)
u_1	-	upstream depth average flow velocity	(m/s)
u_{cr}	-	critical depth-averaged flow velocity for initiation of motion	(m/s)
u_{des}	-	depth-averaged design velocity	(m/s)
y_s	-	maximum scour depth below the upstream river bed	(m)
y(t)	-	time-dependent local scour depth	(m)
α_t	-	empirical coefficient for characteristic time of scour hole development	(-)
Δ	-	relative density	(-)
φ	-	angle of internal friction	(degrees)
σ	-	normal stress	(kN/m^2)
τ	-	soil shear stress	(kN/m^2)

SUMMARY

A Revetment Test Structure has been constructed at Bahadurabad on the left bank of the Jamuna river according to the design presented in Annex 8. This design has been based on the best available knowledge at the beginning of the Project as well as a series of laboratory investigations. As the structure had been conceived as a *test structure*, the possibility of damage and an associated learning from experiences had been anticipated. The performance of the Revetment Test Structure has been monitored for a period of several years with the aim to develop and optimise design criteria and cost-effective construction and maintenance methods, which will serve as future standards most appropriate for the prevailing conditions at the Brahmaputra-Jamuna river and other rivers of Bangladesh.

The monitoring of the Revetment Test Structure is presented in Annex 10. Annex 11 presents the evaluation of the findings from the monitoring period 1997 to 1998. During these years the flow attack was significant. In the following period sedimentation formed a sand bar in front of the structure and reduced the flow attack.

After some comments on the design methodology in Chapter 2 and on the conceptual design in Chapter 3, a description of the hydraulic loads on the test structure is given in Chapter 4. The observed behaviour of the revetment toplayers, launching aprons and falling aprons is explained in Chapter 5. Chapter 6 covers the river response with respect to large-scale morphological changes. Various aspects of the development of the scour hole in front of the revetment structure are described and analysed in Chapter 7. Finally, Chapter 8 gives an overview about the improved design parameters.

1 INTRODUCTION

The protection of riverbanks by revetments has been applied in several ways since generations around the globe. Revetments are usually employed in cases, where minor correction of the river's flow is required to achieve planform stability. These structures allow for a stabilisation of the riverbanks.

A first reconnaissance bathymetry survey at the area of Bahadurabad was carried out in June 1995. The bank was at that time about 500-600 m off the today's location of the Revetment Test Structure. The structure was completed in all respects on June 12, 1997 after location and layout had to be corrected. On June 20, 1997, the test structure came under flow attack for the first time after the protecting cofferdam had been eroded.

2 APPROACH FOR ESTIMATION OF HYDRAULIC AND LOCAL MORPHOLOGICAL PARAMETERS

A general introduction into the design methodology of a river training structure is given in Annex 7. The standard methods to determine the hydraulic and morphologic parameters for the design can also be applied for a revetment. For these reasons reference is made to Annex 7.

It is important to note that the analysis of the data in an overall statistical frame, specifying recurrence intervals, annual maxima and minima, etc. is fundamentally impossible. First of all, the heaviest loads do not occur at the highest discharges, but at discharges near bankfull, under certain morphological conditions. In case of a braiding river, statistics to derive design parameters should not be applied to hydrological data but to morphological data. This differs fundamentally from the usual discharge statistics for flood protection along rivers and usual wave climate statistics for coastal engineering. Only for a meandering river the standard methods can be applied in principle. Furthermore, the presence of structures and the associated scour holes make the approach conditions to a structure more severe than would be expected from a statistical analysis of the conditions in a natural river without structures (see summary of Annex 1). For a sound statistical analysis of this phenomenon, the available data on morphological conditions near structures in the Jamuna are insufficient.

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3 BASIC DESIGN CONCEPT

Bank protection works can generally be divided into structures which reduce the hydraulic loads on a bank and structures which increase the resistance against bank erosion. The design concepts for both types of works differ in some respects. The Revetment Test Structure in Bahadurabad is of the type which increase the resistance against erosion.

Because the concentrated flow along a revetment can experience a sudden widening at the downstream end, a downstream termination is generally necessary. The discontinuity in cross-sectional area generates highly turbulent flow causing deep scour. The same phenomenon is to be considered at the upstream end for an upstream termination. Further comments on the conceptual design of a river training structure are given in Annex 7.

4 EVALUATION OF HYDRAULIC LOADS FOR THE DESIGN

4.1 INTRODUCTION

The attack of the river on the Revetment Test Structure is characterised by the maximum hydraulic loads viz. water level, flow velocity and wave height. To evaluate the hydraulic loads, measurements have been carried out at the test structure. The water levels have been measured at two staff gauges on a daily basis throughout the whole year. The results are described in Section 4.2.

For the measurement of the flow velocities different methods were applied. The results are evaluated in Section 4.3. Depth averaged flow velocities have been measured by Valeport velocity meter (Section 4.3.1) in the vicinity of the test structure, where access with a boat was possible, and additionally, float tracking was carried out (Section 4.3.2). The field measurements including the flow pattern (Section 4.3.3) are compared to the Physical Model Tests (Section 4.3.4). Finally, the estimation of roughness coefficients is described (Sections 4.3.5).

The wave heights have been estimated only during storms and windy days in the channel by visual observation from boat. They were also investigated at the water level gauging stations. Details can be found in Section 4.4.

4.2 WATER LEVELS

4.2.1 Monitoring

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 moved from Section A to Section B on August 27, 1997 because the water levels in Sections A were not representative and a too limited range of the gauge had been used. The downstream gauge was placed in Section H with a distance of about 550 m to the upstream gauge. The reading of the water level at the staff gauges was normally scheduled at 8 a.m., 1 p.m. and 5 p.m. daily. In general, the readings from 8 a.m. were used as a daily representative value presented in hydrographs. The average accuracy of the reading has been estimated to ±0.01 m. During certain phases of the monitoring period the downstream gauge could not provide data. It had been damaged due to sudden rise of the water level and was reinstalled afterwards.

A third water level gauge was installed by Hydroland, a local survey firm within another project, at the left bank of the Jamuna river near Bahadurabad railway ghat in May 1998 for additional information. This gauge was located 1641 m upstream from the gauge located at Section B of the test site. The water level was read daily from May 28 to July 13, 1998. The levels were compared with the water levels read at the upstream gauge at the test structure (Fig. 4.2-1). The graph shows that the water levels at these gauges are consistent. This indicates that these measurements are accurate.

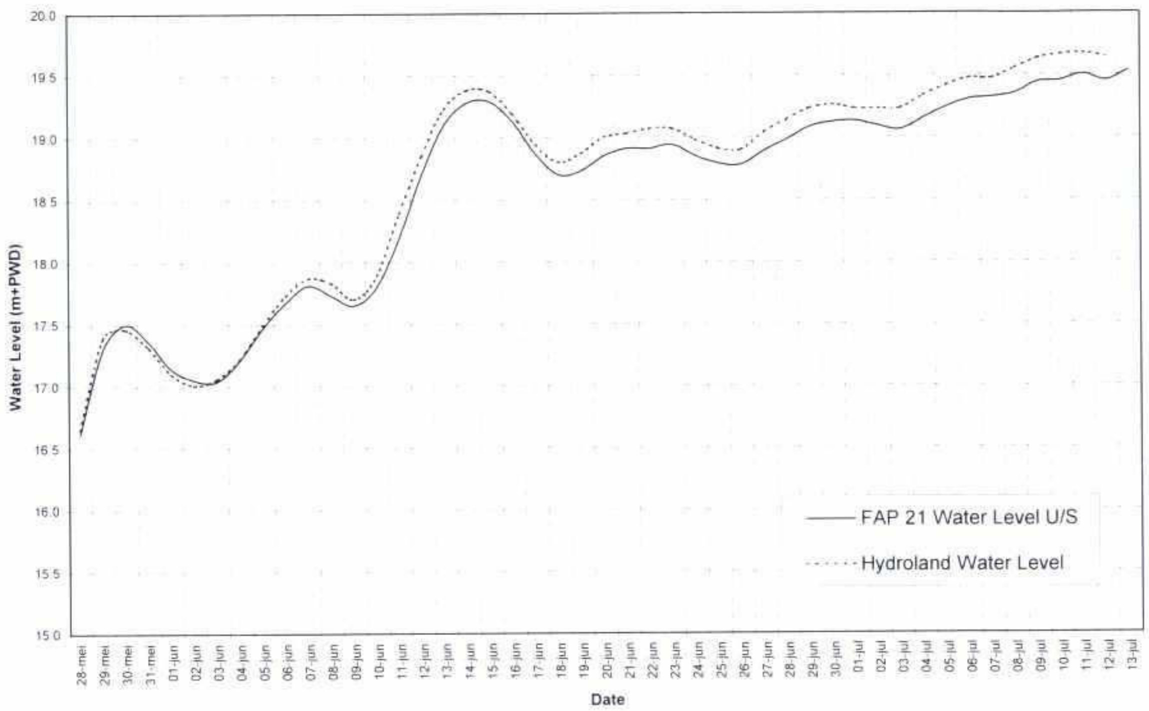


Fig. 4.2-1: **Water levels measured upstream from the railway ghat and at the test structure 1998**

4.2.2 Peak Flows

The hydrographs recorded at the upstream gauge are presented for the years 1997 and 1998 in Fig.4.2-2. To evaluate them statistically, the minimum, maximum and average water levels from the period 1994 until 1998 are included. Data before 1990 were recorded from a BWDB gauge located at about the same position. The difference in location varied due to bank erosion. However, the locations were very close to each other when projected on the thalweg. The maximum water level of 20.61 m+PWD occurred in the period before 1990.

However, a definition of the return period of the hydrographs in 1997 and 1998 can not be given, even with taking into account the shown statistical frame. The statistical investigation appears not to be relevant due to the reasons mentioned before. The main activity of the channel morphology is not necessarily dependant on the highest water level. It could be observed that bank erosion occurs mainly during the monsoon season at bankfull flows. The sediment load of the channel might be highest at water levels far below the highest water levels. This illustrates the difference between the whole river and a single channel.

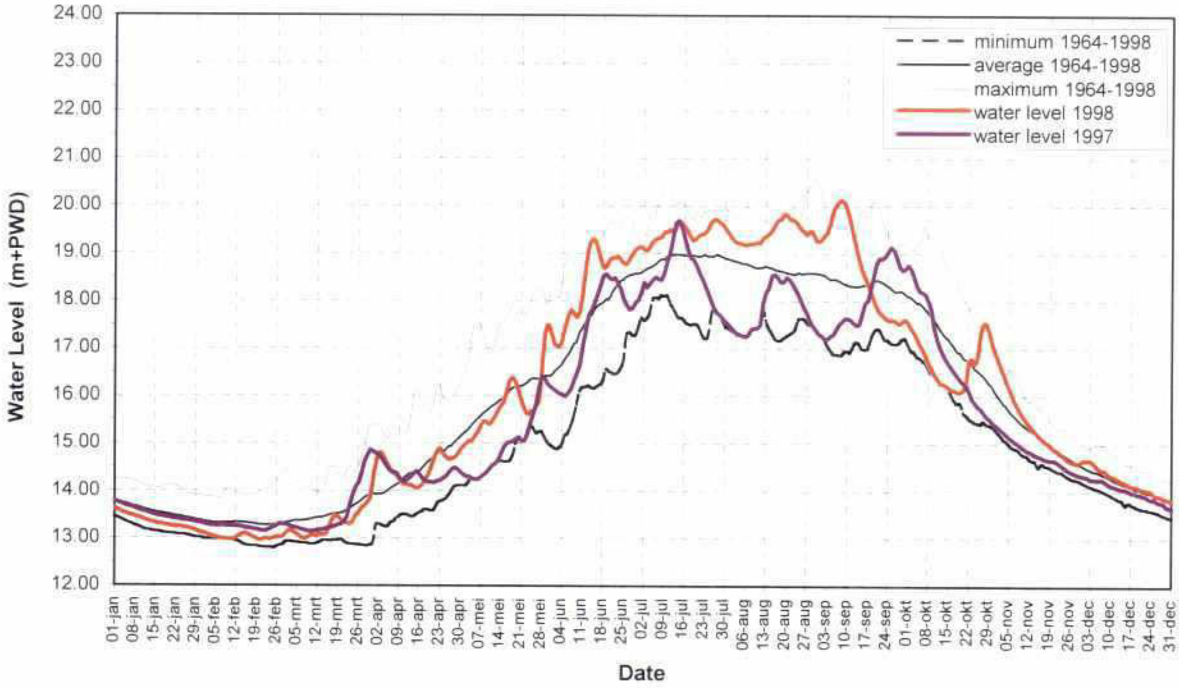


Fig.4.2-2: Hydrographs in 1997 and 1998 in a statistical frame

The hydrographs show 4 peaks during the 1997 monsoon and 5 peaks during the 1998 monsoon with maximum water levels above 18 m+PWD (see Table 4.2-1). The highest recorded level in this period of 20.12 m+PWD occurred on August 20, 1998. During the receding limb of the 1998 hydrograph an unexpected local peak water level at 17.53 m +PWD had been measured on October 27/28, 1998, probably because an upstream barrage had released discharge (see Fig. 4.2-2). However, all recorded water levels were below the design water level of 21.10 m+PWD (25 years return period). The lowest water level of 12.96 m+PWD was recorded in February 1998. A general trend about the development of high and low water levels in the future cannot be deduced due to the limited recording time.

Year	Date	Time reading	Upstream gauge	Downstream gauge	Estimated return period
		hours	m+PWD	m+PWD	Years
1997	June 19	8.00	18.56	-	< 2
	July 14	8.00	19.69	19.55	< 2
	August 16	8.00	18.56	18.48	< 2
	Sept. 25	8.00	19.14	19.05	< 2
1998	June 15	8.00	19.29	19.26	< 2
	July 14	8.00	19.69	-	< 2
	July 26 and 27	8.00	19.71	19.69	< 2
	August 20	8.00	19.82	19.05	< 2
	September 8	8.00	20.12	19.90	< 2

Table 4.2-1: Maximum water levels in 1997 and 1998 (the given return periods are approximate)

The duration curve of the water levels (see Fig. 4.2-3) shows that the mean water level in 1998 was higher as compared to 1997 and the previous two years. At 17.3 m+PWD the lower part of the floodplain started to be inundated, whereas the upper part of the floodplain had a level of about 18.5 m+PWD. The duration curve of the water levels in 1998 shows that the water level remained above this bankfull level for an extremely long period of about 3 months (see Fig. 4.2-3). During this period the floodplain had been inundated almost permanently.

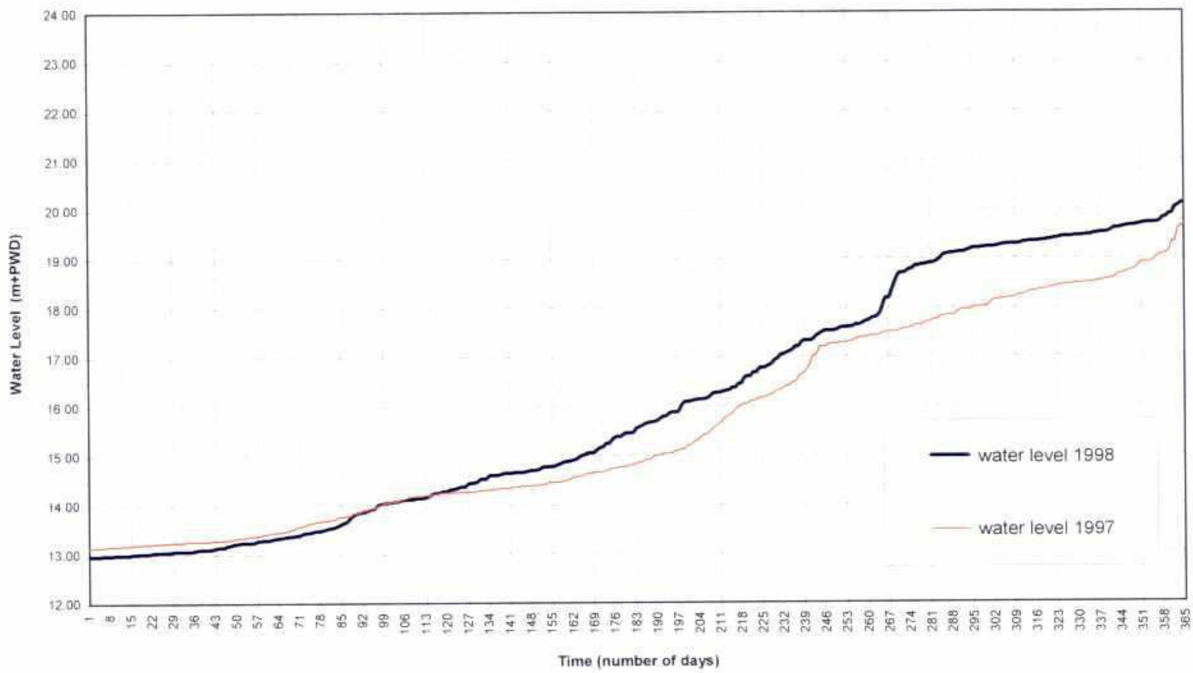


Fig. 4.2-3: Duration curves of non-exceeded water levels for 1997 and 1998

4.2.3 Rate of Rise and Fall

The daily increase or decrease of the water levels, which were measured at the upstream and the downstream gauge (see Fig. 4.2-4 and Fig. 4.2-5 respectively) are very important regarding the stability of the bankline (slides). During the rising limb of the peak flows in 1998 the water level had raised at the upstream gauge more than 0.7 m/day. The maximum fall was recorded to be about 0.28 m/day. In comparison with previous years these maximum rise and fall values were extreme. In November the rise and fall in the receding limb of the hydrograph are normally very small, but in 1998 they were considerable because a barrage had located upstream released water.

382

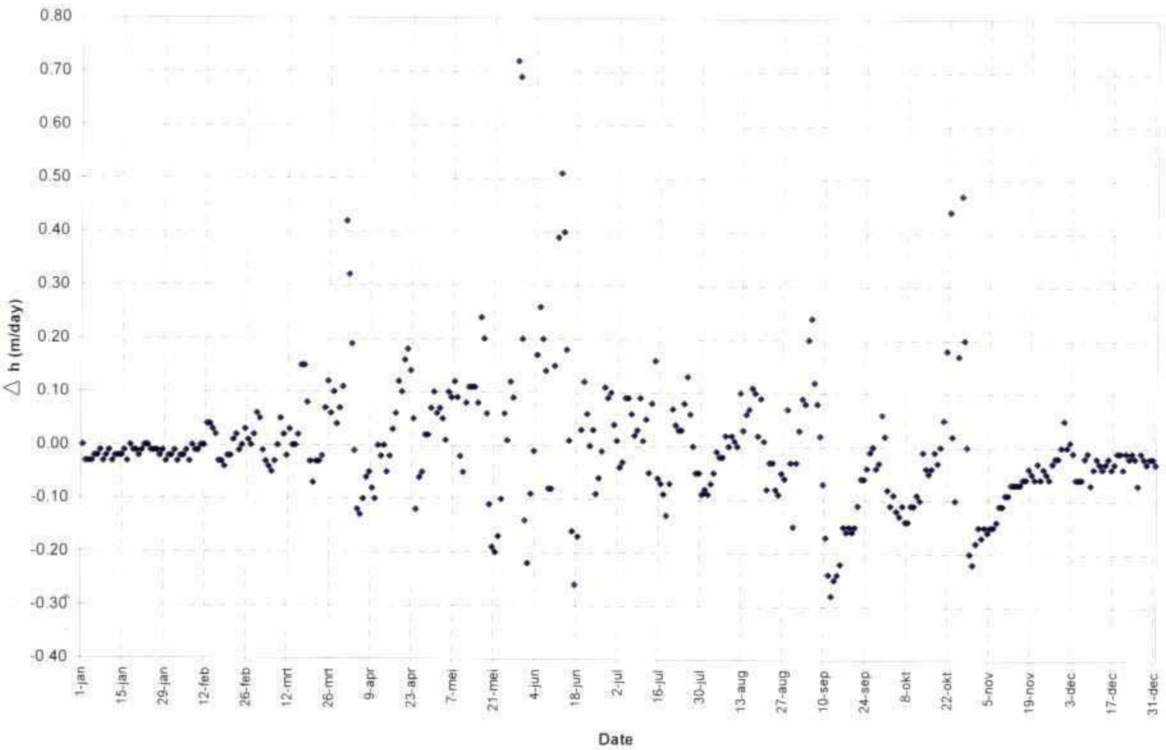


Fig. 4.2-4: Daily rise and fall of water level at the upstream gauge 1998

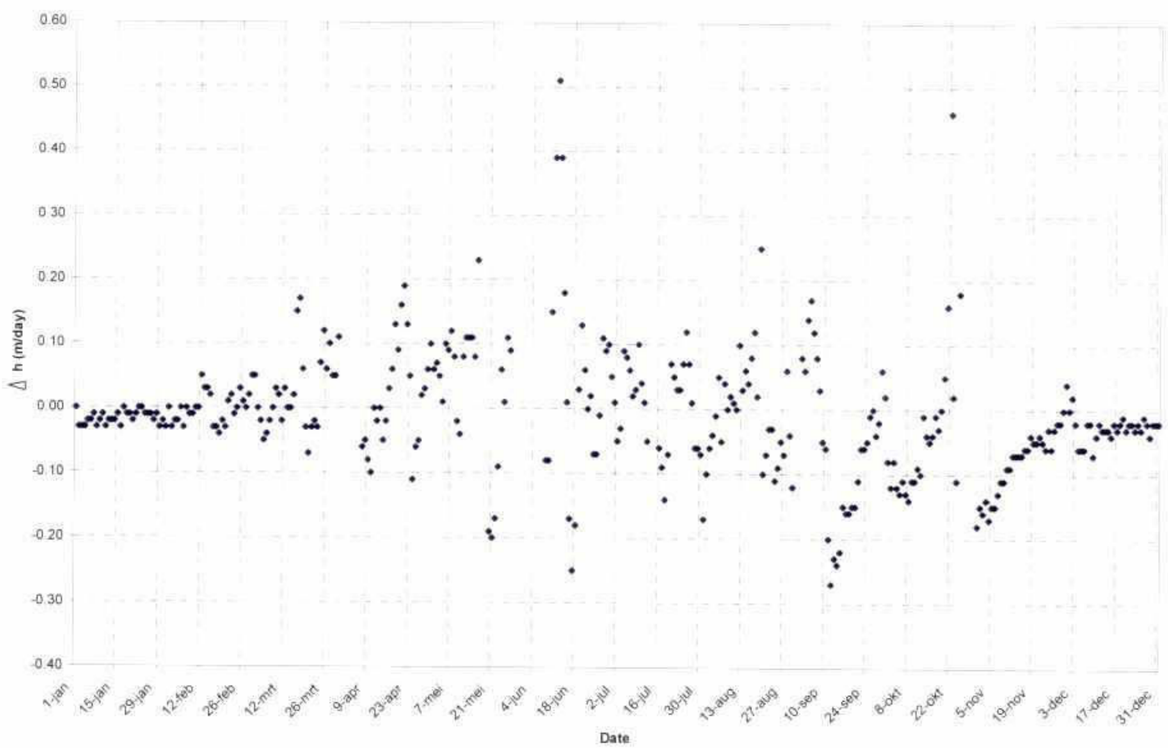


Fig. 4.2-5: Daily rise and fall of water level at the downstream gauge 1998

The rise and fall of the water level at the upstream gauge have been analysed in more detail using the readings at 8 a.m., 1 p.m. and 5 p.m. to determine the water level rise and fall in m/hour instead of m/day. The rise and fall varies normally between 0.01 m/hour rise and 0.01 m/hour fall (see Fig. 4.2-6). During peak flows higher values were observed (0.04 and 0.02 m/hour respectively). Surprisingly a very quick rise of nearly 0.16 m/hour occurred during the rising limb of the 1998 hydrograph. The water level increase was 63 cm within 4 hours and produced the daily rise of more than 0.7 m/day in Fig. 4.2-4.

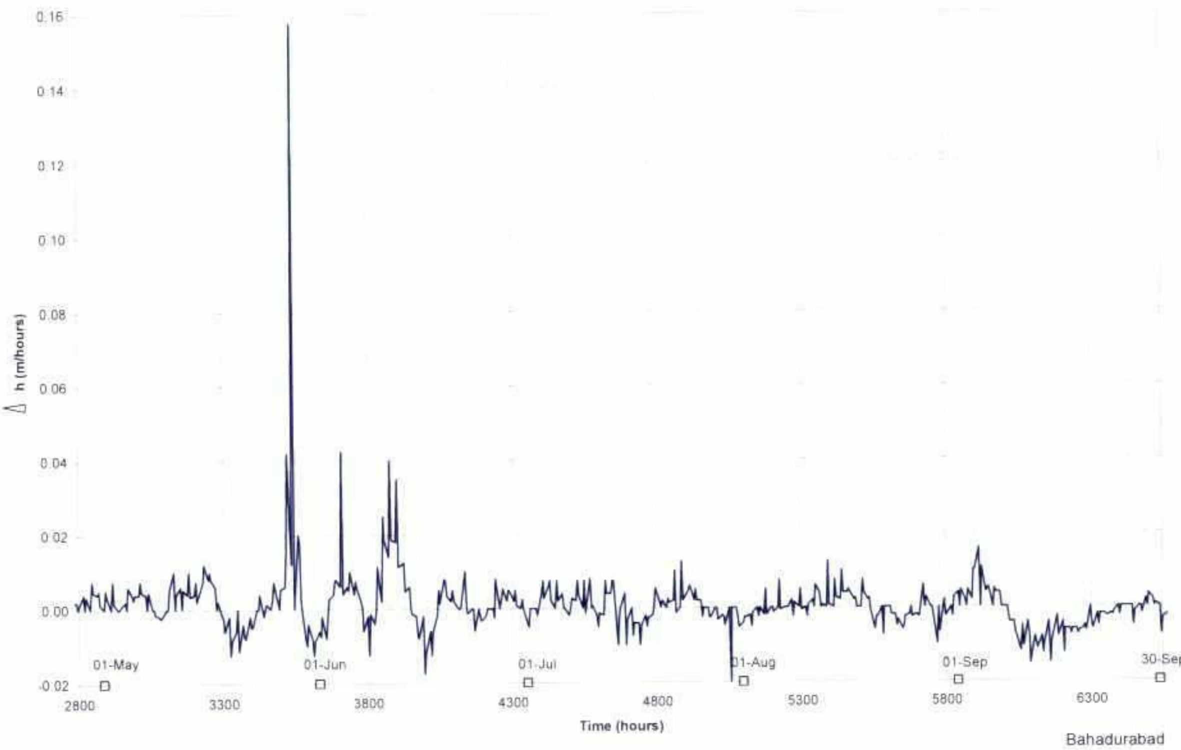


Fig. 4.2-6: **Rise and fall of the water level in [m/hour] at the upstream gauge in summer 1998**

4.2.4 Water Level Gradient

The water level gradient was calculated from the water levels at Hydroland gauge and the gauges in Sections B and H of the test structure. The water level gradient in front of the test structure shows for 1998 very small variation except August and the first half of September 1998, (see Fig. 4.2-7). Although the results are within the range of the average gradient (between 0.03 m/km and 0.10 m/km), the gradients were very steep in August 1998 (up to 0.40 m/km). In this month the area in front of the test structure had been silted up to water depths of only 1 to 3 m. In September this area had been eroded and a deep channel developed again in front of the test structure (see also Section 7).

The theoretical necessity that the water level gradient is related to the daily rise or fall of the water level has not been confirmed by the data as can be seen in Fig. 4.2-8. It might be that the morphological developments were dominant and that they had obscured this relationship.

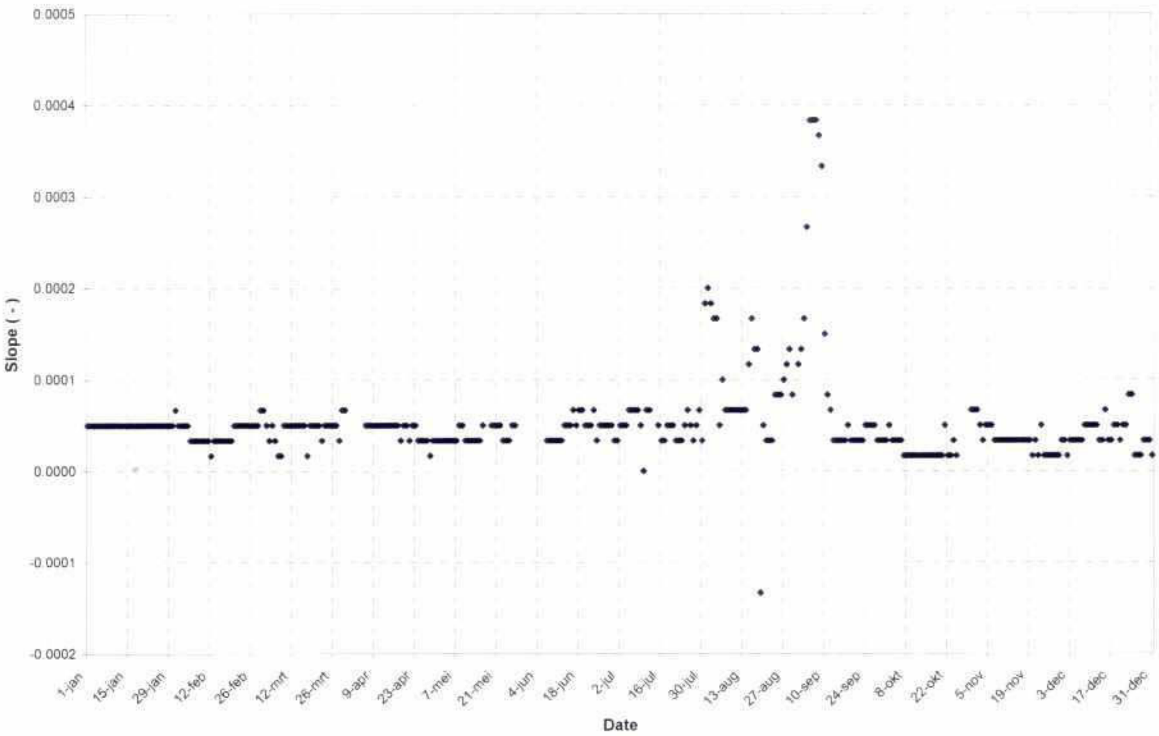


Fig. 4.2-7: Water level gradient in 1998

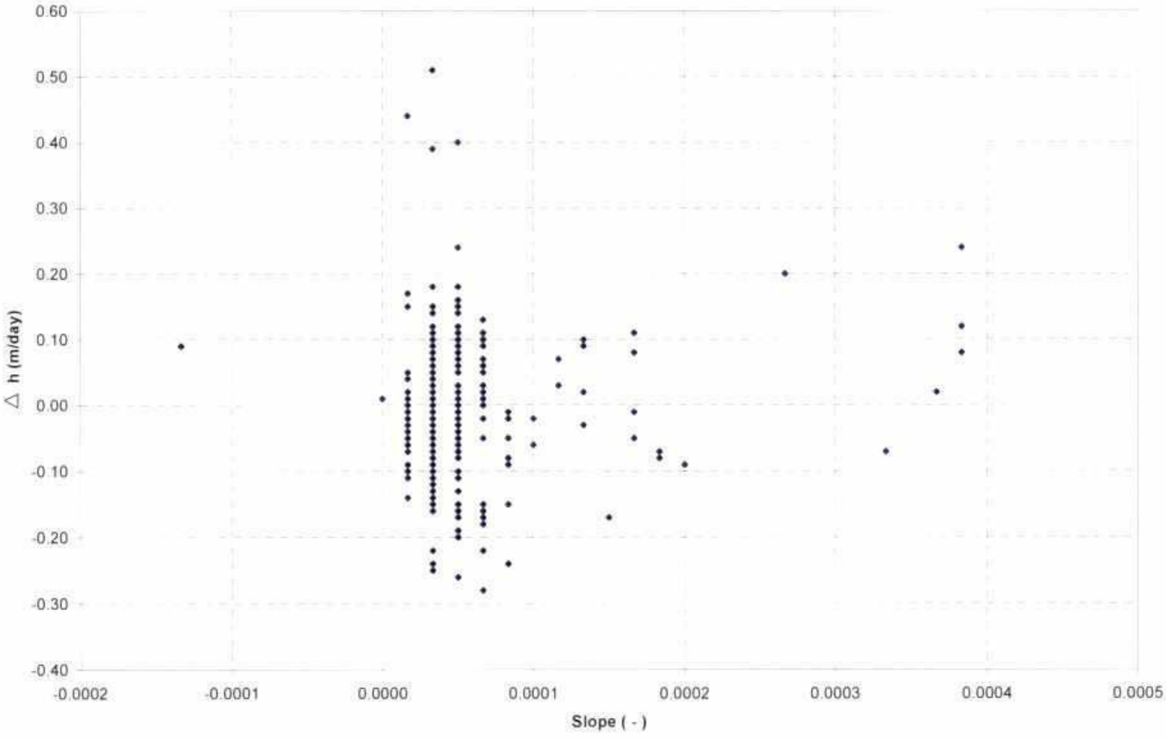


Fig. 4.2-8: Water level gradient as function of the water level rise and fall 1998

In Fig. 4.2-9 the results from the different pairs of gauges are compared. It shows that the water level gradient between the Hydroland gauge and the gauge at Section B is about 0.02 to 0.05 m/km higher than the gradient measured by the upstream and downstream gauges at the test structure.

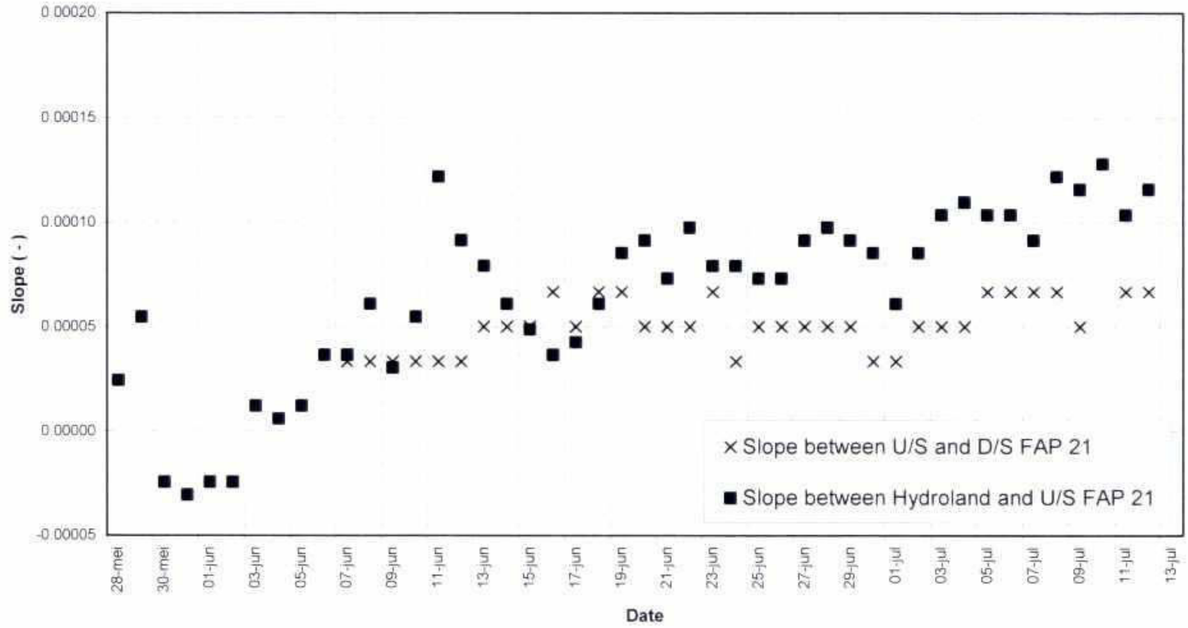


Fig. 4.2-9: Water level gradients upstream from and in front of the test structure from May until July 1998

The water level gradient during the monsoon is presented as function of the water level in Fig. 4.2-10. This graph shows that the water level gradient was higher before the peak on July 14, 1997 than after this peak for the same water level. This tendency is clearly shown because the hydrograph over the 13 days had a single peak. However, it is less clear for other 1997 peakflows because these flows had multiple peaks. The flood wave had a steep front followed by the recession limb with a lower gradient. The rise and fall of the water level obviously shows the same quality as the water level gradient. Both investigations show that the water level gradient in the Jamuna river can vary considerably at short distance and with time. This might be the result of the morphology of the channel in the observed period.

280

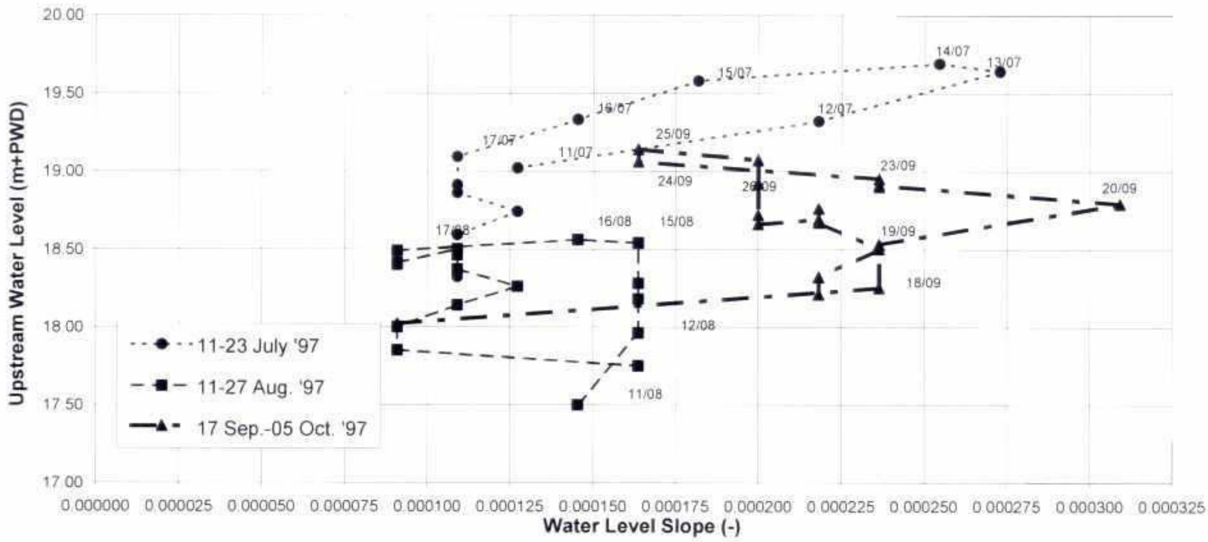


Fig. 4.2-10: Water level gradient as function of the water level in 1997

The morphological development might explain the unexpected variation in water level gradients. Initially, some doubt existed about the accuracy of the water level readings, but it has been checked several times and no inaccuracy had been found. After the extremely steep gradients developed during the highest peak flow of the 1998 monsoon, the area eroded quickly 4 to 6 m in the first half of September. In that period most char areas silted up, if no new channel developed through them.

It was observed that temporarily the water level gradient can increase to 4 times the average due to the development of a cut-off in front of the test structure.

4.3 FLOW VELOCITIES AND FLOW PATTERN

4.3.1 Valeport Measurements

The flow velocities above the river sided limit of the falling apron have been measured by Valeport velocity meter during special surveys. Measurements were carried out up to 120 m from the crest of the revetment into the direction of the main channel. In Fig. 4.3-1 a plan with the location of Valeport flow measurements and the numbering of the flow velocity verticals is given. Fig. 4.3-2 shows the standard cross-section in the middle of the test sections. The shallow water verticals were located 50 to 90 m and the deeper verticals 100 to 120 m from the crest of the revetment.

The velocities have been measured by Valeport velocity meter only during one day in the 1997 monsoon (September 11, 1997) but regularly in 1998.

At the observed day in 1997 the depth averaged flow velocities were rather low compared to the simultaneously measured maximum values of 3 to 4 m/s. Average velocities were calculated in the range 0.67 m/s to 1.29 m/s (Fig. 4.3-3). The standard deviation of the slowly accelerating flow along the edge of the falling apron varied from 6 to 9 % of the local depth averaged flow velocity in Sections B to G and from 20 to 30 % at the downstream termination in Section H (see Table 4.3-1), flow separation occurred.

Fig. 4.3-1: Locations and numbering of flow velocity verticals

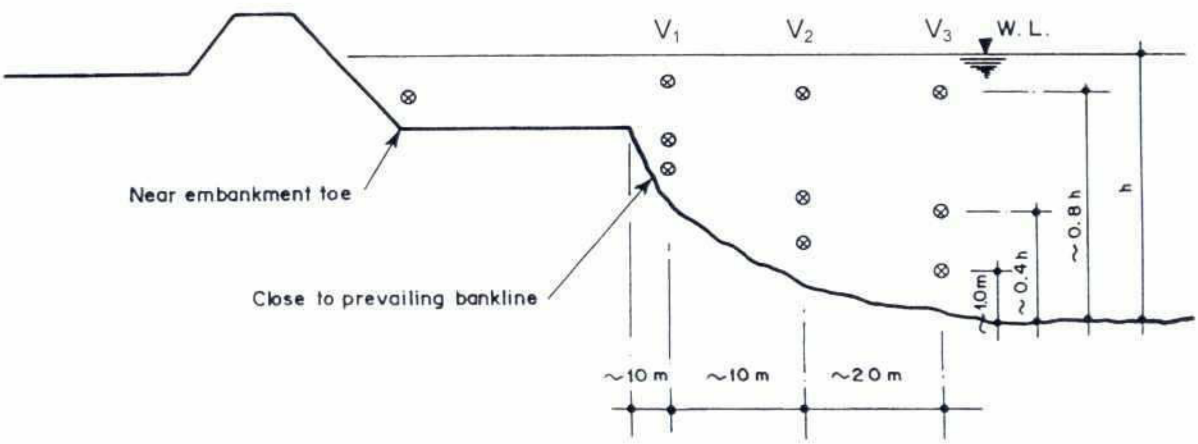


Fig. 4.3-2: Position of flow velocity measurement

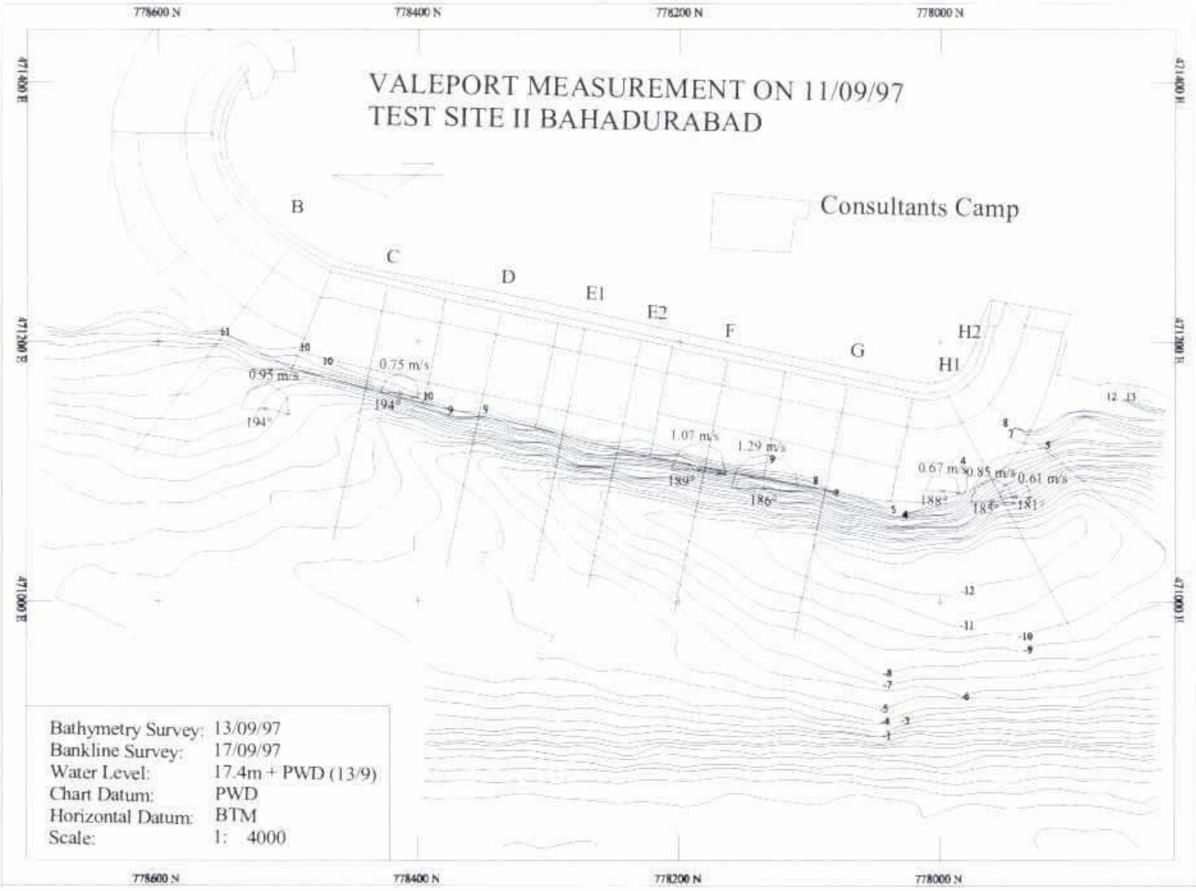


Fig. 4.3-3: Depth averaged flow velocities on September 11, 1997

Date	Section	Vertical 1*)	st.dev	Vertical 2 *)	st. dev.	Vertical 3 *)	st.dev
May 29	B	0.22	0.09	0.63	0.06	0.89	0.05
	C	0.75	0.07	0.92	0.05	0.83	0.03
	D	0.99	0.04	1.04	0.03	0.85	0.02
	E1	0.66	0.06	1.02	0.04	0.93	0.02
	F	0.76	0.05	0.94	0.03	0.78	0.03
	G	0.79	0.07	0.99	0.04	0.93	0.05
	H1	0.42	0.09	0.84	0.09	1.11	0.03
June 8	G	0.69	0.07	1.01	0.06	0.97	0.03
June 22	E1	0.52	0.07	1.00	0.14	**2.02**	0.10
	F					**2.12**	0.09
Sept.15	E1	0.98	0.07	1.17	0.06	1.16	0.05
	G	0.86	0.09	1.30	0.05	1.55	0.06
Oct. 31	B	0.85	0.05	0.94	0.05	1.07	0.04
	D	1.59	0.06	1.68	0.07	1.48	0.09
	F	0.86	0.15	1.32	0.07	1.48	0.09
	H1	0.21	0.06	1.25	0.08		

*) Verticals 1, 2 and 3 stay in a row perpendicular to the revetment; vertical 1 is closest to the revetment and vertical 3 is closest to the channel, see Fig. 4.3-1.

Table 4.3-1: Depth averaged flow velocities (m/s) and standard deviations (m/s) above the face of the falling apron in 1998

The maximum depth averaged flow velocities measured during the 1998 flood season were 2 to 2.1 m/s on June 22 (see Table 4.3-1). They are below the design flow velocity of 3.5 m/s. The measurements in Sections B to H1 recorded on May 29, 1998 at an average water level of 17.72 m+PWD are shown in Fig. 4.3-4.

The maximum measured values of the standard deviation decrease from 0.15 m/s in the shallow verticals to 0.10 m/s in the deeper verticals. The average standard deviation was 4 to 6 % of the local depth averaged flow velocity. Closer to the revetment this percentage is not a function of the averaged flow velocity, but probably a function of the roughness of the face of the falling apron.

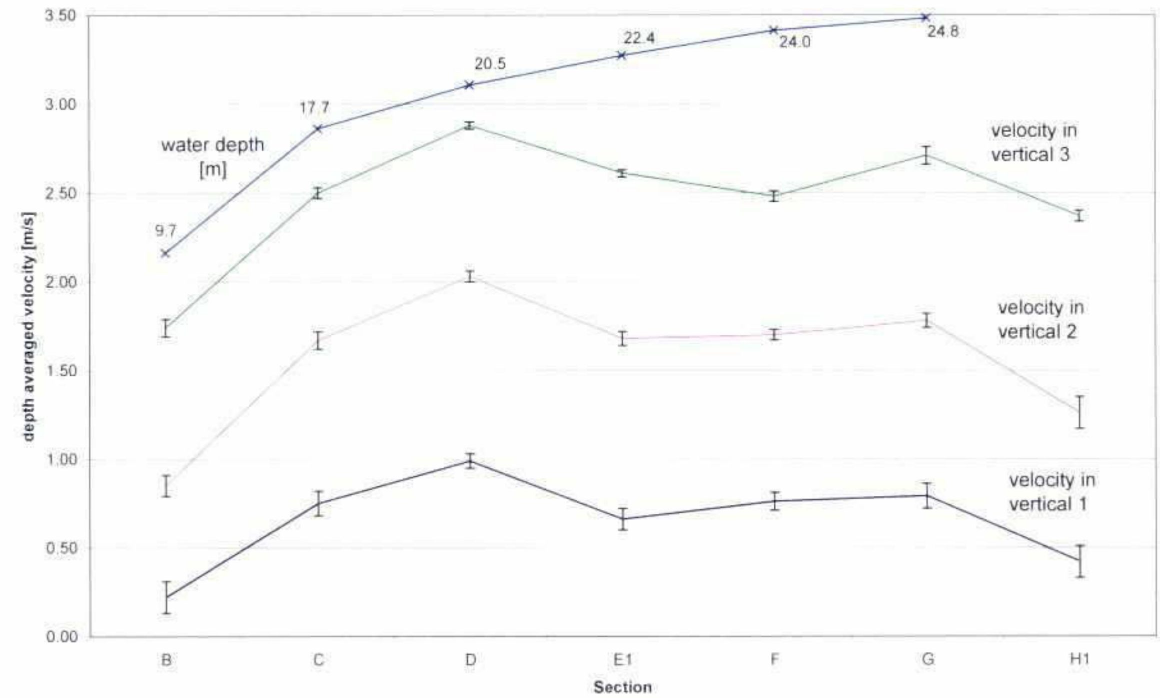


Fig.4.3-4: Depth averaged flow velocities with standard deviation along the test structure on May 29, 1998

4.3.2 Float Tracking

Float tracking had been carried mainly by the University of Applied Sciences, Bremen, which has been described in another detailed report (Consulting Consortium FAP 21/22, April 1999). In regions of particularly low water depth, i.e. above the launching apron, additional measurements using simple floating devices were made necessary. These measurements, performed by the FAP 21 monitoring team, utilized benchmarks at the centrelines of Section B to G as reference points to calculate the flow velocity on basis of the travelled distance and time elapsed.

The flow velocities above the **launching apron** were measured almost daily during 1997. The float tracking showed maximum flow velocities (up to 2.10 m/s) on August 12, 1997 when the water level raised for the third peak in the hydrograph. These measurements have been selected because they gave the highest recorded flow velocities (Fig. 4.3-5). Due to the small amount of data it can not be proved how these values relate to the maximum values possible.

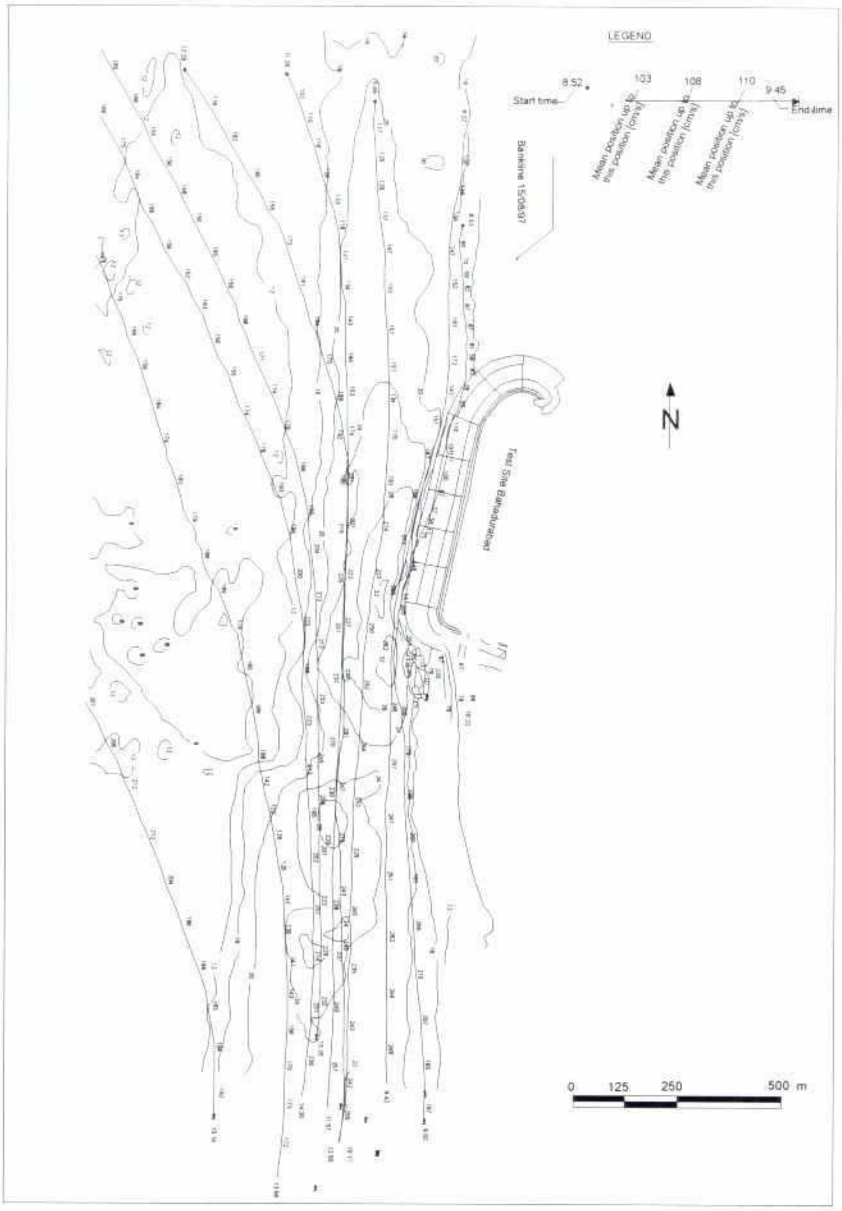


Fig.4.3-5: Example of float tracking on August 12, 1997

Above the **launching aprons**, the flow velocities increased in downstream direction due to increasing protrusion of the structure and the distraction of the stream lines (see Table 4.3-2).

Section	Maximum 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 4.3-2: Maximum flow velocities measured above the launching aprons by float tracking in 1997

The maximum flow velocities above the face of the **falling apron** were measured on July 13, 1997 (see Table 4.3-3). This survey was made just one day before the flood had reached its highest water level in Bahadurabad, see also hydrograph in Fig. 4.2-2.

Section	Maximum Flow velocity (m/s)			
	surface	-3 m	-6 m	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 4.3-3: Maximum flow velocities measured above the falling apron by float tracking on July 13, 1997

Above the falling apron a sharp gradient of the flow velocities in downstream direction exists. The floats were sensitive to turbulent action as eddies and vortices but the general tendency is that the flow velocity is accelerated as soon as the main channel changes its course due to the revetment. Along the edge of the falling apron the flow velocity gradually increases.

The maximum flow velocities have been measured above the downstream scour hole, see example of the float tracking in Fig. 4.3-1. The maximum measured surface flow velocity is 3.9 m/s, the maximum depth averaged flow velocity above its deepest point is 2.7 m/s. This is also below the depth averaged design flow velocity of 3.5 m/s.

Fig. 4.3-6 shows the velocity profile obtained from different measurements in Section E1 and G between June and September 1998. The measurements are scattered around the logarithmic flow velocity profile, which was assumed for the design flow velocity distribution under ideal conditions. The actually measured data show the influence of other effects, e.g. changing roughness, changing water depth and oblique flow attack. However, the assumed theoretical velocity distribution provides a good approximation as basis for the structural design.

The depth averaged flow velocity along the revetment slope for the same data is given in Fig 4.3-7. The assumed dependency between flow velocity and the distance from the toe of the revetment slope (see Annex 8, Subsection 2.4.5) is well represented by the measurements, even the cross-section differed from the assumed design cross-section.

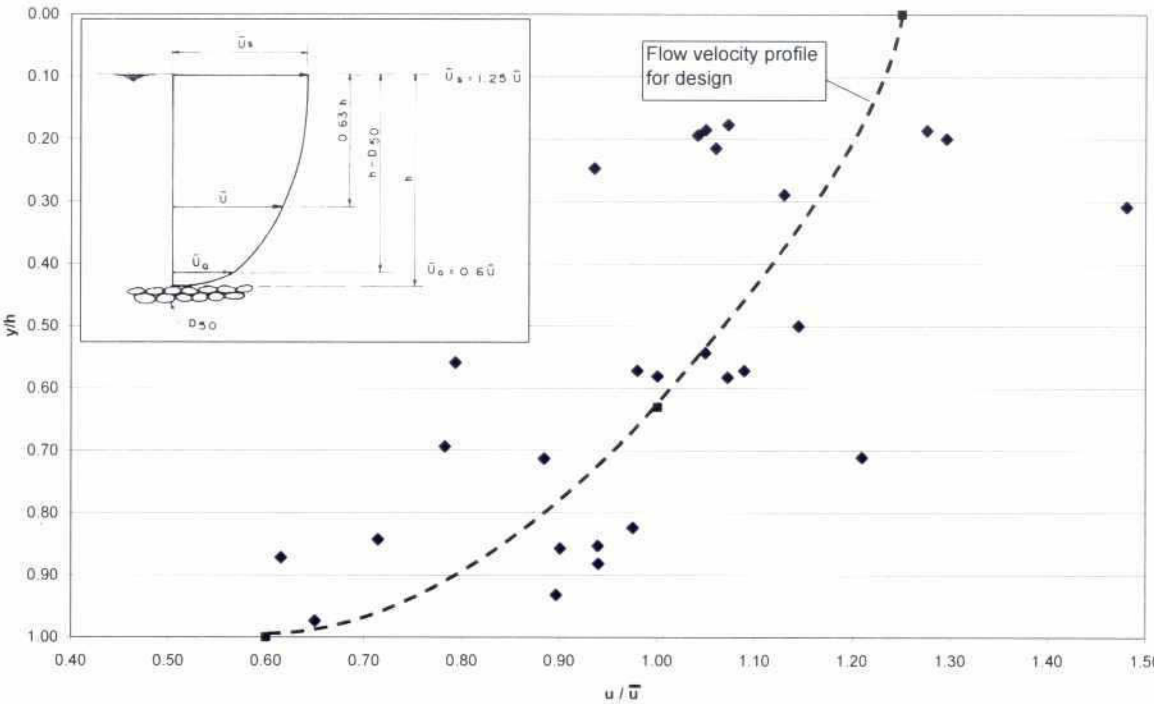


Fig. 4.3-6: Measured flow velocities at different cross-sections and design velocity profile

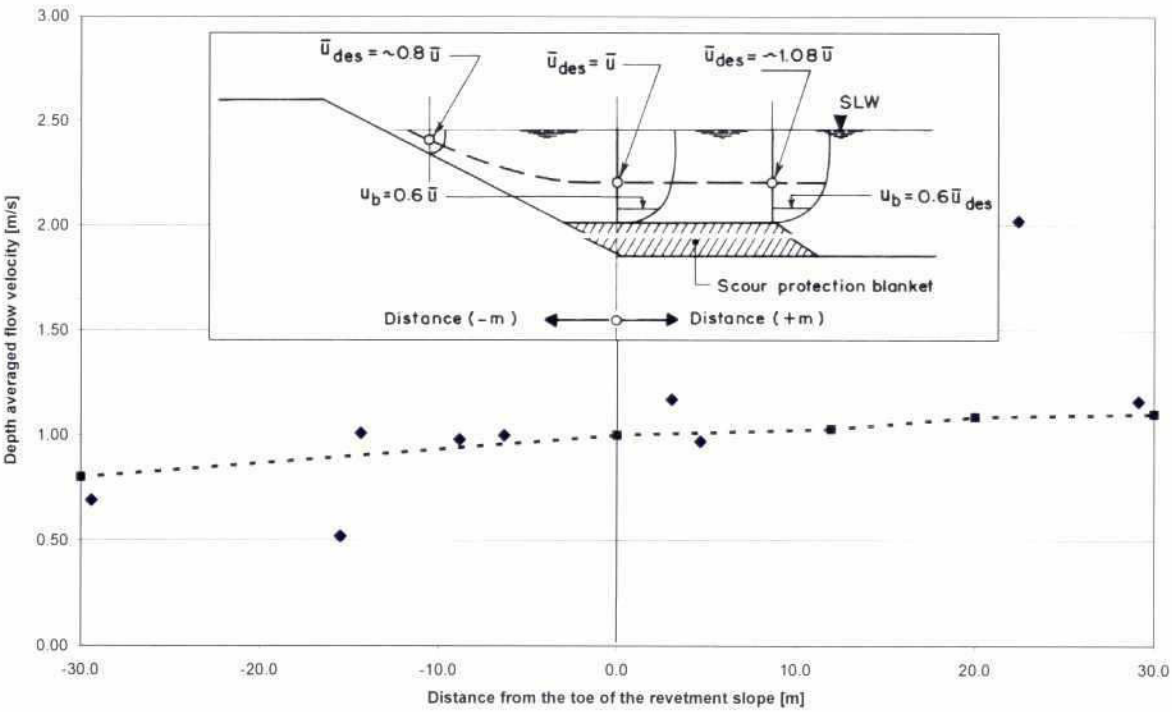


Fig. 4.3-7: Calculated depth averaged flow velocities from measurements and design velocity distribution along the revetment slope

4.3.3 Flow Pattern

Float tracks from the 1998 monsoon (data from June 14 and 15, June 24 to 28 and July 10 to 14, 1998) have been combined with the depth averaged flow velocities determined from Valeport measurements close to the test structure (data from June 07, June 22 and August 04, 1998 respectively). The Fig. 4.3-8, Fig. 4.3-9 and Fig. 4.3-10 show the development of the bathymetry (surveyed on June 14 to 16, June 25 to 27 and July 11, 1998 respectively) along with the depth averaged flow velocities. They indicate a deep scour hole at the downstream side of the test structure. In July it had been silted up more than half.

The transverse gradient of the flow velocities in front of the revetment increases gradually from upstream to downstream. This means that the maximum flow attack is on the downstream end of the test structure. However, during the first peak flow on June 14 and 15, 1998 the pattern in the transverse gradient was different. The gradient became more gentle from Section C to Section D as the flow entered a deep scour hole in front of the revetment.

The maximum flow velocities in the channel were measured just above 2 m/s in these 1998 surveys. In 1998 the flow velocities in the channel were considerably below the maximum flow velocities observed during the previous monsoon. This means that all measured flow velocities in 1998 have been also below the design flow velocities. A recurrence interval for the measured flow condition cannot be stated because not enough data for a sound statistical analysis is available.

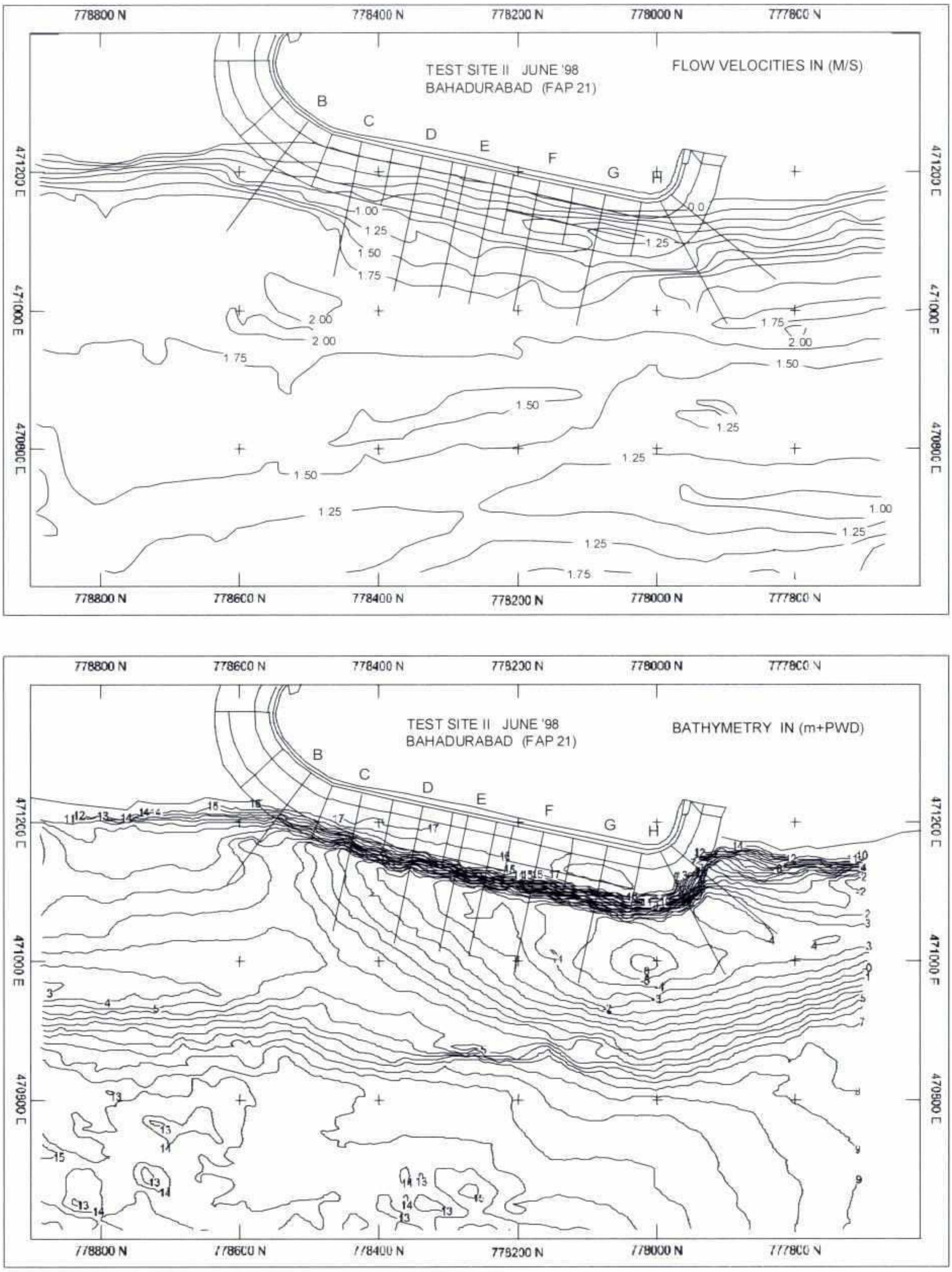
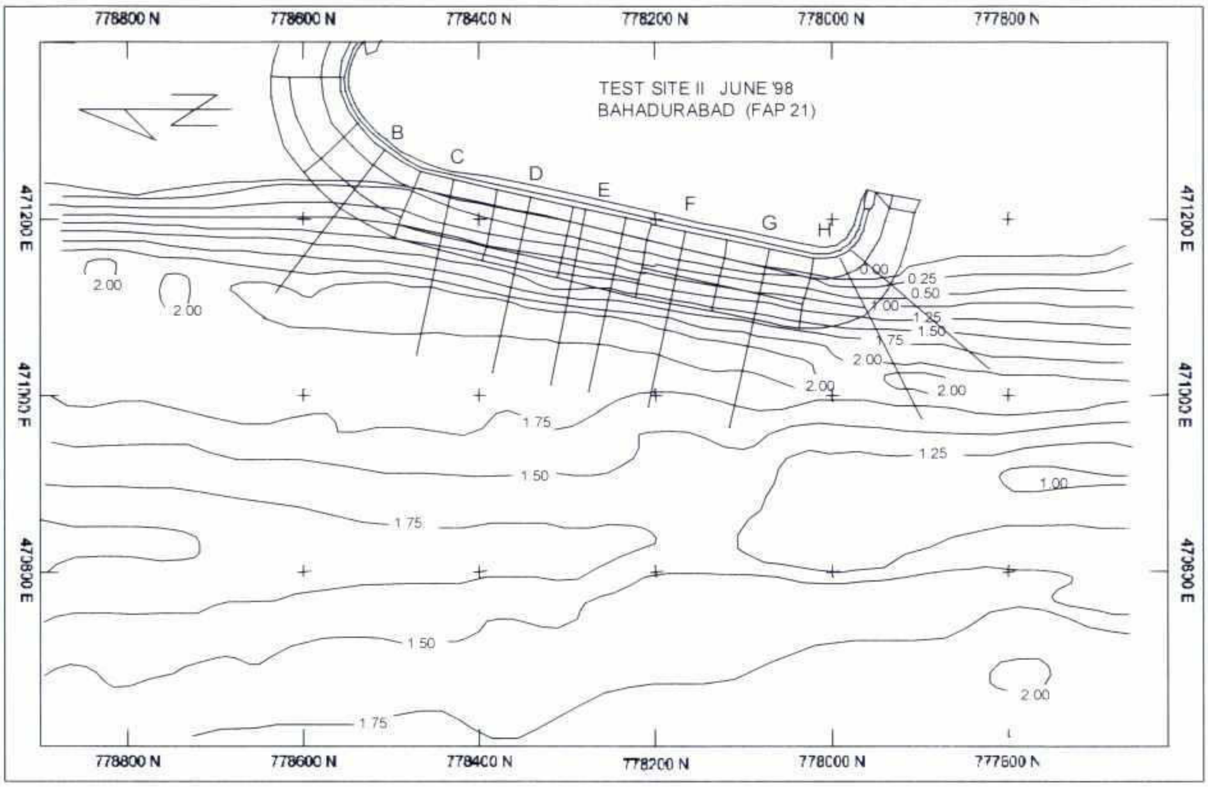


Fig. 4.3-8: Flow pattern and bathymetry during June 07 to 16, 1998



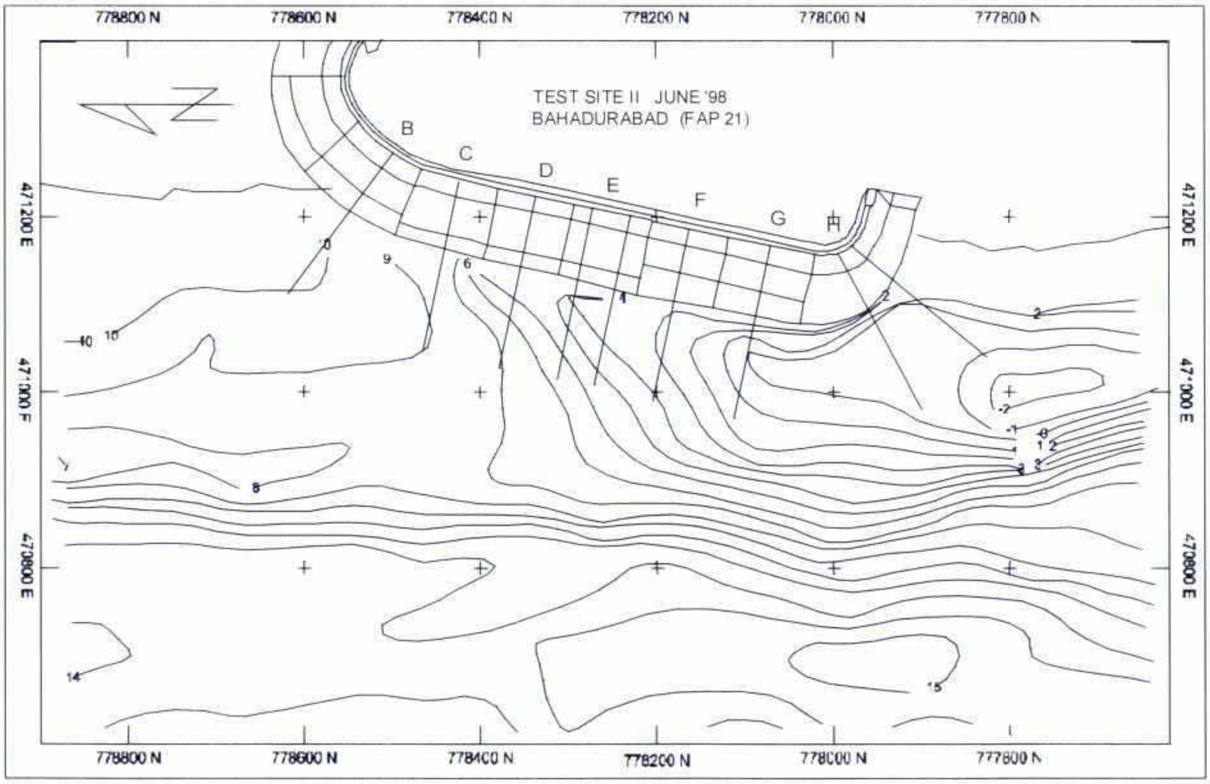
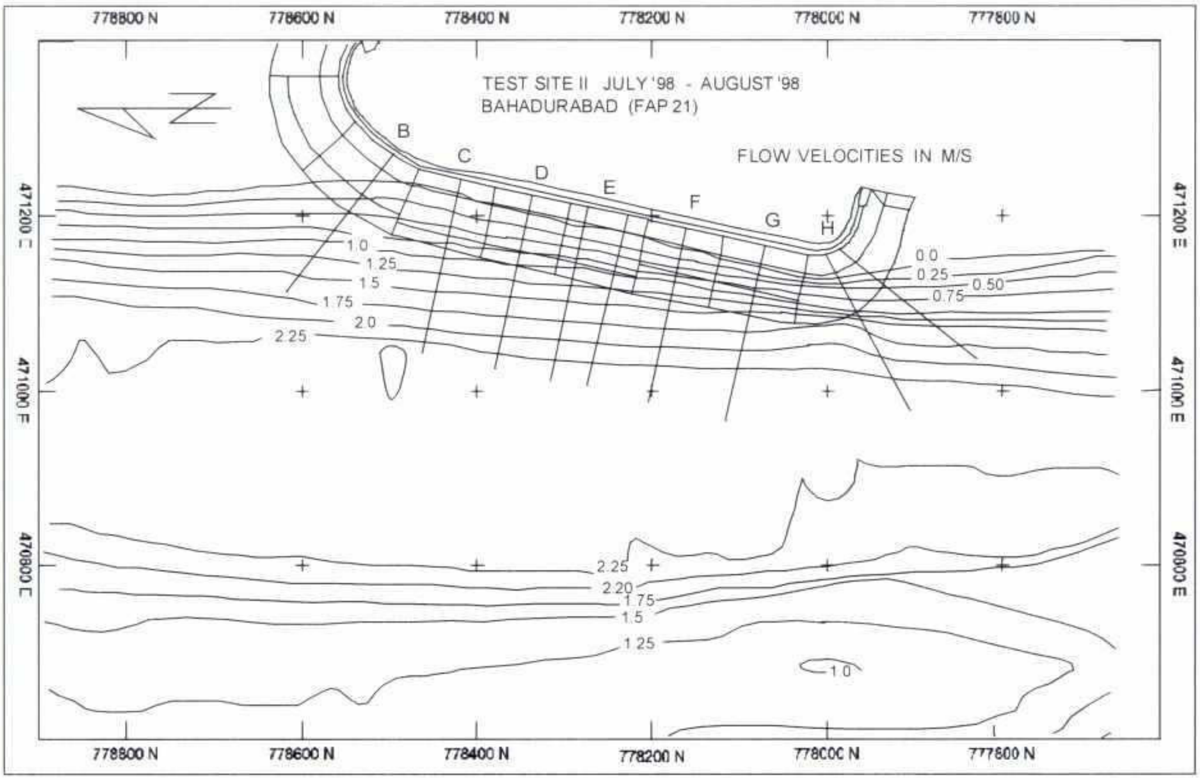


Fig. 4.3-9: Flow pattern and bathymetry during the period June 22 to 28, 1998



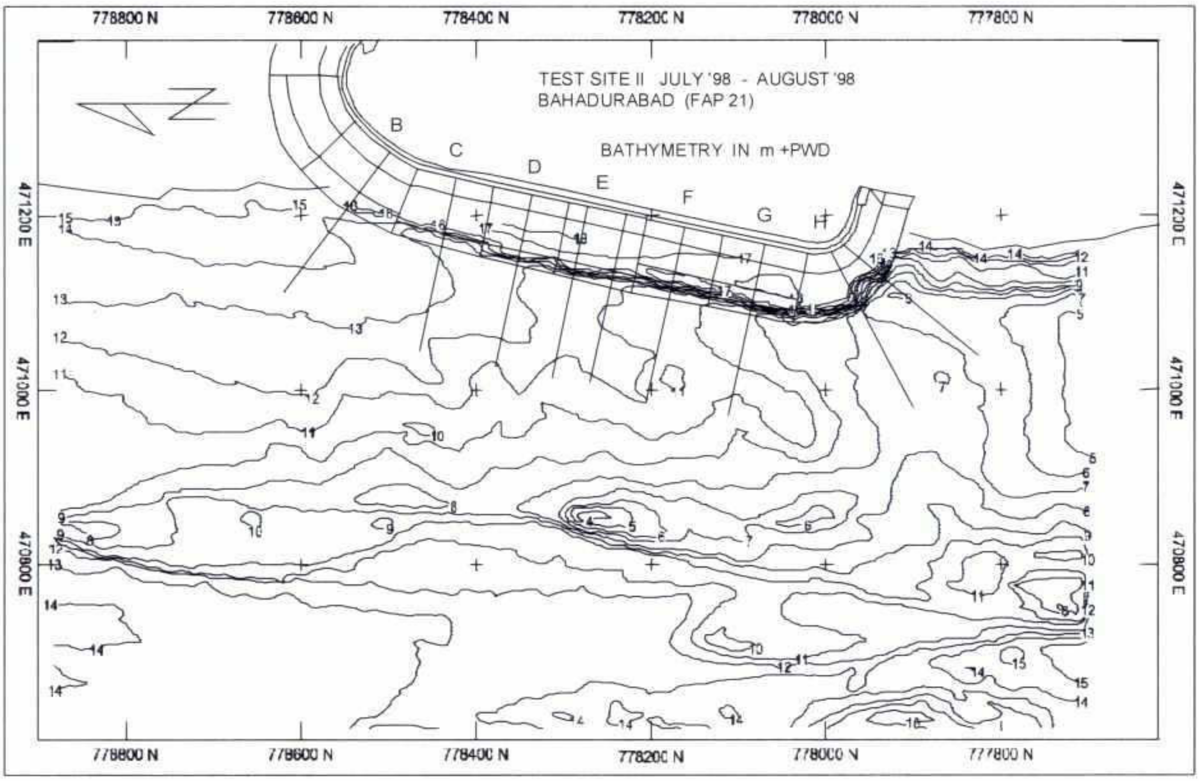


Fig. 4.3-10: Flow pattern and bathymetry during the period July 10 to August 04, 1998

4.3.4 Comparison with Physical Model Tests and Design Flow

In additional model investigations maximum flow velocities of 4.4 to 5.7 m/s had been estimated for a return period of 25 years. Compared with the maximum velocities in Table 4.3-3, these flow velocities seem to be overestimated, as already mentioned in the report on this model investigation. However, part of these differences can probably be explained from the different approach channels 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).

During the design phase it had been planned to dredge the underwater slope of the revetment structure and a constant flow velocity distribution in transversal direction had been assumed. Later it was decided to construct a falling and launching apron just above standard low water level, therefore it was assumed that the maximum flow velocity will occur above the deepest point at about 120 m from the crest of the revetment. Besides this, after the actual flow attack on the revetment begun, the falling and launching apron started to operate and its material had partly fallen, so that the actual surface conditions deviated from the assumed smooth cover layer. Therefore, a direct comparison of measured and expected velocities is not straightforward. However, in general the flow velocity distribution in transversal direction along the structure front was well reproduced by the method applied (compare Fig. 4.3-7).

4.3.5 Estimation of Roughness Coefficients

With varying water level gradients, as described before, an estimation of the flow velocity by the Chézy formula is very difficult and is dependant on the time. During high flow the Chézy formula reads:

$$u = C \cdot \sqrt{h \cdot I} \tag{4.3-1}$$

in which

u	=	flow velocity	(m/s)
C	=	Chézy coefficient	($m^{0.5}/s$)
h	=	water depth	(m)
I	=	water level gradient	(-)

To estimate the Chézy coefficient, the depth averaged flow velocity and the water levels information at both staff gauge locations have to be measured at the same time. This was provided only in 1998. Consequently, only the distributions of the depth averaged flow velocities in Section G on June 08, 1998 as well as in Section E-1 on June 22, 1998 and September 15, 1998 (Section 4.3.2) could be used to estimate the roughness coefficients of the sloping face of the structure and the adjacent riverbed. The average water level gradient is estimated for the same days from the upstream and the downstream water level gauge. The gradient on June 08 was about 0.03 m/km, and on June 22 and September 15 the gradient was 0.05 m/km (see Fig. 4.2-7).

A good fit between the measured (Valeport measurements) and calculated flow velocities has been obtained by selecting the hydraulic roughness k_s to be 0.03 m for the river bed and 0.5 to 1 m for the face of the falling apron (see Table 4.3-4). These values correspond to Chézy coefficients C of about 70 $m^{0.5}/s$ for the riverbed and 40 to 45 $m^{0.5}/s$ for the face of the falling apron.

229

Survey date in 1998	Section	Gradient (m/km)	C, riverbed ($m^{0.5}/s$)	C, sloping face ($m^{0.5}/s$)	k_s, riverbed (m)	k_s, sloping face (m)
June 08	G	0.03	72	40 - 45	0.03	1.00
June 22	E-1	0.05	69	40 - 45	0.03	1.00
September 15	E-1	0.05	-	40 - 45	-	0.50

Table 4.3-4: Empirical roughness of sloping face and adjacent riverbed from measurements in 1998

In general, a high hydraulic roughness of the face of the falling apron contributes to the decrease of the flow velocities from the middle of the channel towards the bankline. Due to the partial articulation of the falling apron, the area of high roughness became larger, which therefore had a stability effect on the toe protection. The roughness experienced by the flow is produced by the roughness of the elements (for example cc-blocks) and the unequal surface of the sloping face as the falling process develops differently in the various cross-sections.

4.4 WIND AND WAVES

4.4.1 Wind Speed and Wind Direction

The wind speed and the wind direction were measured once daily. As both parameters were not measured continuously during the day, the recorded values are not necessarily representative for the whole day. Still they give an impression how the wind speed and direction vary during a year.

Most of the time the wind speed did not exceed a value of about 2 to 3 m/s (see Fig. 4-4.1). During the 1997 monsoon six stormy days have been observed with a maximum wind speed of 12 to 13 m/s. In 1998 the highest wind speed was 15 m/s, measured in April (before monsoon season).

Eastern winds dominate during the monsoon period, therefore waves did not break on the revetment test structure. A correlation between wind direction and speed could not be found. The wind direction during 1998 in Bahadurabad varied significantly from the wind directions in Kamarjani, which is probably due to the more or less sheltered measurement locations.

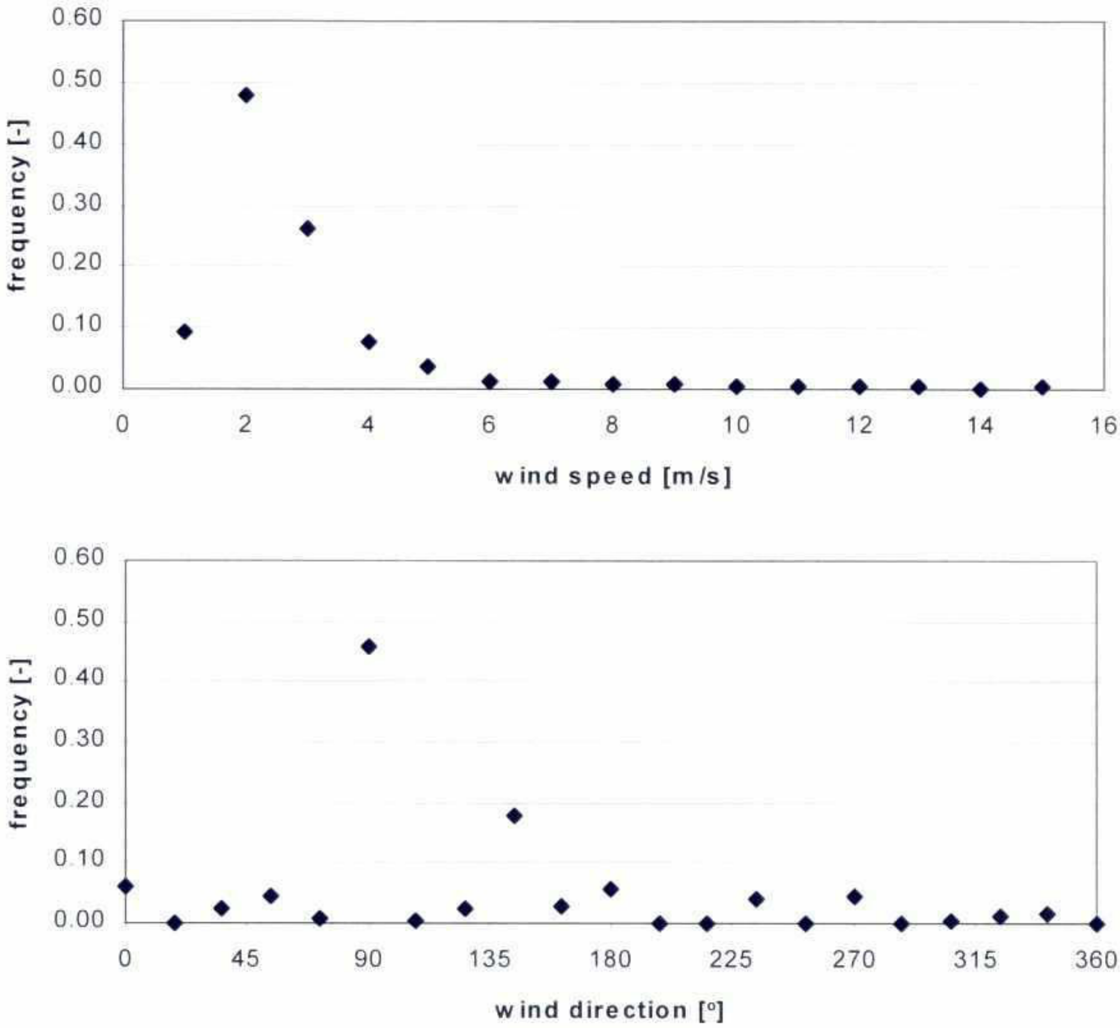


Fig. 4.4-1: **Frequency of wind speed and direction in 1997 and 1998**

4.4.2 Waves

The survey team estimated 1 m high waves in the channel 200 m from the revetment on 11 July 1998, when the South Eastern wind had a speed of only 1 m/s. A few days later, on July 22 1998, they estimated 1.5 m high waves in the channel. This was higher than the design wave height of 1 m close to the revetment. However, visual wave height observation without gauges often leads to overestimates.

In 1997 a maximum wave height of 1 m has been estimated on July 03, 1997, during the monsoon when the wind direction was east. Most of the time in 1998 the wave height was less than 0.15 m at the water level gauges (see Fig. 4.4-2). The wave gauges stay in small embayments upstream and downstream from the revetment, consequently the wave heights might be damped. A correlation between wind speeds and wave heights could also not be noticed from the recorded data. Wave runup on the revetment has not been measured, but it was observed that the wave attack on the structure was very small.

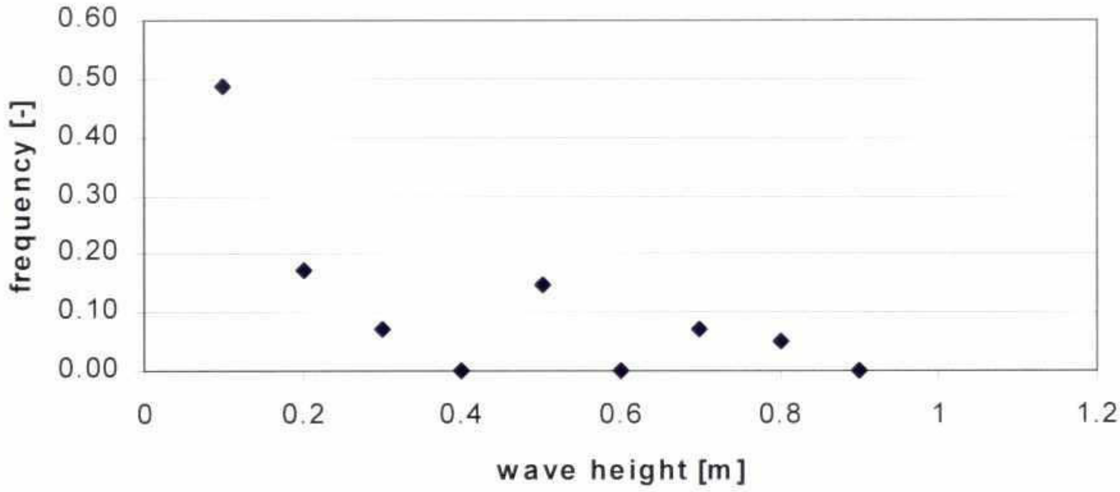


Fig. 4.4-2: Frequency of wave height in 1997 and 1998

730

5 EVALUATION OF STRUCTURAL BEHAVIOUR

5.1 COVER LAYERS AND FILTER LAYERS

5.1.1 Damages

The design of the test structure can be evaluated on the basis of the observations in the different sections of the structure. During the 1997 monsoon, the first flood after completion of the construction of the Revetment Test Structure, limited deformations were observed in Section E. Rain cuts and bulging of the ship-lap interlocking concrete slabs at about 2 to 3 m below the water level due to accumulation of migrated sand had been observed. The depths of the cuts were 0.15 to 0.2 m. Heavy rainfall had been recorded during the preceding months in Bahadurabad (on June 15 as well as on September 19 and 20, 1997, 65 and 51 mm rainfall respectively). After this rainfall also some settlements were noticed in the upper revetment in Sections E and F above the water level line (see sketches in Fig. 5.1-1).

During the 1998 monsoon the revetment slope was stable and without any deficiency except Section E where the interlocking blocks have shown some settlement. Some interlocking blocks have been displaced due to heavy rainfall (140 mm in June). At the transition between Sections E and F some settlement was going on. Besides the rain cuts, the flood flow caused some interlocking blocks to settle and to move on September 11, 1998.

5.1.2 Physical Explanation

The soil cuts induced by water run-down on the revetment slope had been observed already action in the filter test investigations under simulated wave (1994). A critical slope of the structure face had been determined.

The flow phenomena due to rain and due to wave run-down are similar near the actual water level. However, no significant wave action during the monsoon was observed at the Revetment. Therefore, the recorded heavy rainfall had most probably caused the observed damage.

The pre-fabricated ship-lap interlocking slabs suffered from production irregularities, such as variable shape regarding the roundness of their corners, the thickness of the slabs and variable gaps between the slabs. Insufficient and non-uniform surface contacts existed between geotextile filter and slabs, thus allowing down-slope migration of soil particles (piping) due to severe rain penetration.

In subsoil A, which is sandy silt type ML to CL, pipes can develop until a complete failure takes place. Subsoil B, which is silty sand type MS, is characterised as a very dangerous soil because of its relatively low permeability, its relatively high compressibility and its low shear strength. In the filter test investigation, segregation of soil fractions and piping have been observed as the main failure mechanism. These pipes develop below the geotextile, where insufficient contact pressure exists between filter and protective layer.

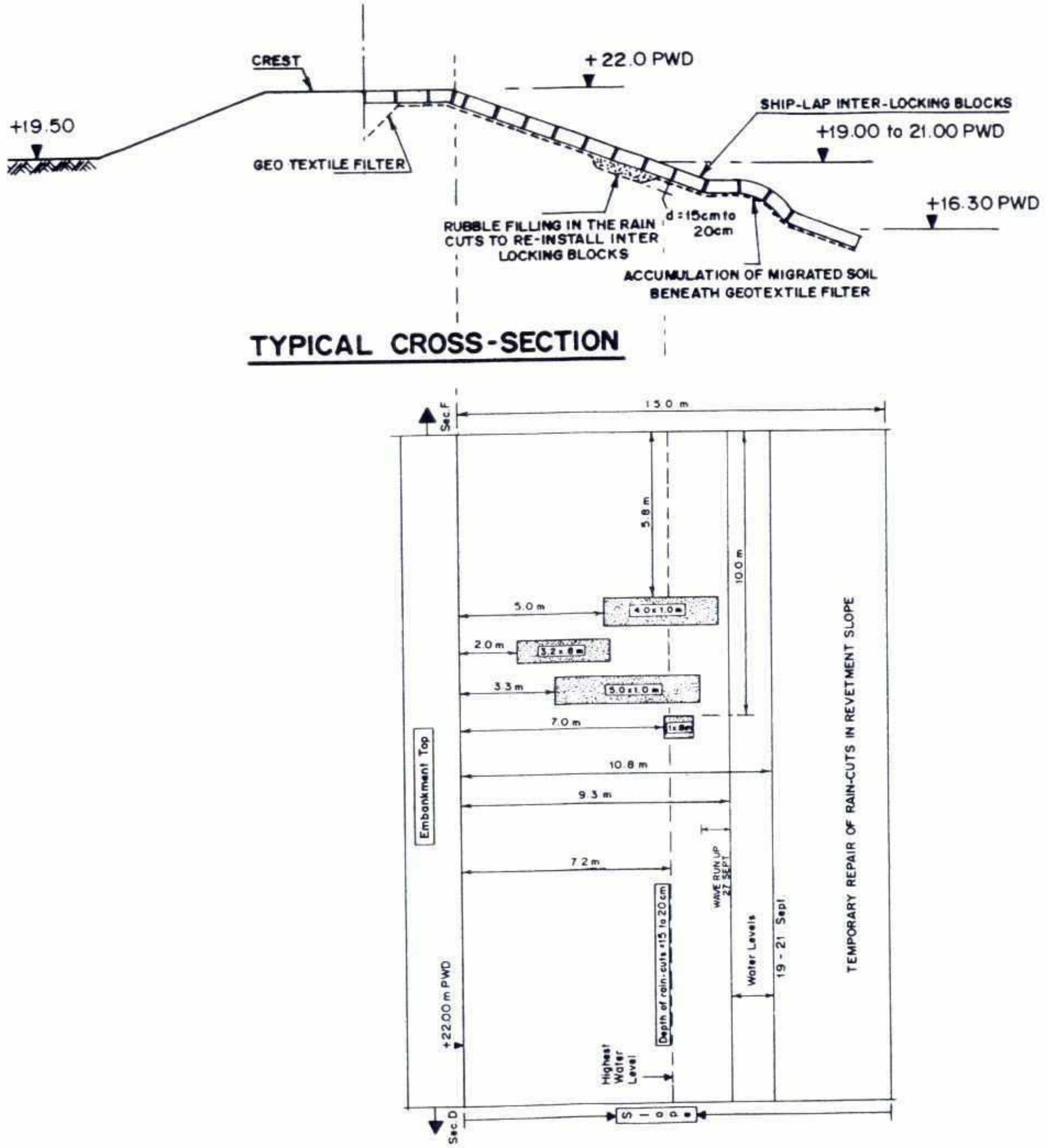


Fig. 5.1-1: Rain cuts under layer of concrete slabs in Section E

The analysis of the tests in the filter rig showed that in case of small and unequally distributed weight from the critical gradient of the geotextile is reduced from 0.5 to 0.2 for subsoil A (see Table 5.1-1). In case of the observed damage at Section E, beam action between the concrete slabs has probably reduced locally the gravity pressure on the geotextile to zero, which might have reduced the critical slope below the actual embankment slope 1V : 3H.

Construction type	Critical slope (-)	
	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 5.1-1: Critical gradients from filter test investigations

It had been observed that the cc-blocks in Section D were stable. The submerged weight of these cc-blocks is higher than the weight of the boulders. Without beam action the weight of these blocks is equally distributed over the geotextile and the critical gradient is between 0.5 and 0.6, which is higher than the actual gradient. Therefore, no damage occurred.

5.2 LAUNCHING APRON

Due to the efficiency of the falling aprons the launching aprons were not subjected to erosion. Only the edge of the articulating Reno mattresses in Section C started to launch early in the 1997 monsoon season (see Photo 5.2-1 and 5.2-2). On July 11, 1997 an erosion of 1.5 m had been observed. The mattress system did perform well. Due to sharp bending, the wiremesh boxes lacing wire of a few cages broke up and some boulders fell out. This is to be seen as an executional short fall. The Reno mattresses articulated over a length of about 8 m (perpendicular to the structure crest) under a steep slope of 1V:0.9H. It is concluded that Reno mattress systems are well applicable, but the proper closing of the wiremesh boxes must be carried out with highest attention.

The flow velocities above the launching apron were analysed in Section 4.3. It can be concluded that all launching aprons have resisted a flow velocity of at least 2 m/s. The observed deformations are negligible as expected, because the design flow velocity for launching aprons was 2.8 m/s.

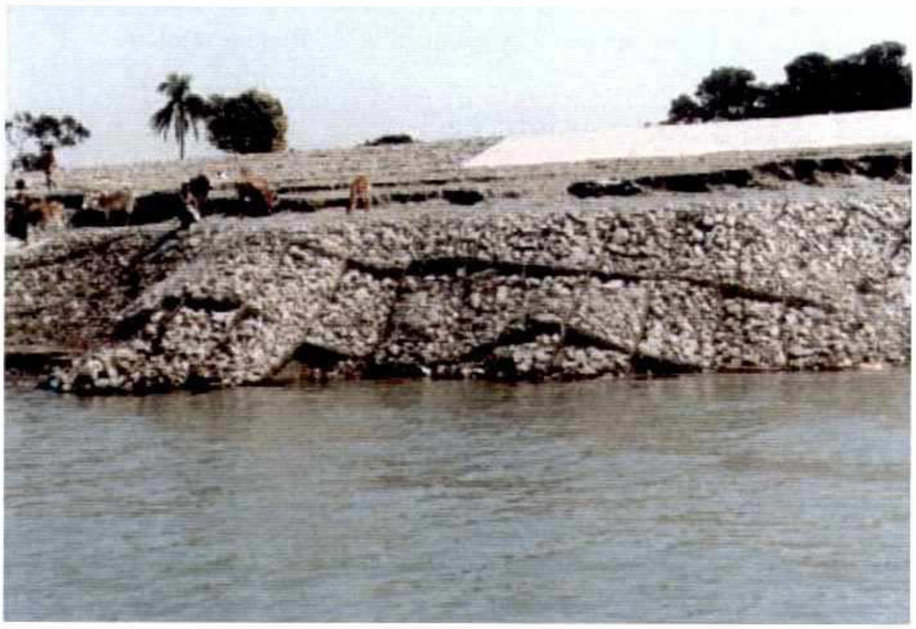


Photo 5.2-1: Section C: Launching apron on February 18, 1998; Water level at 13.00 m+PWD

Photo 5.2-2: Transition Section C/D on February 18, 1998; Section C: Launching apron; Section D: Falling apron; Water level at 13.00 m+PWD

5.3 FALLING APRON

5.3.1 Monitoring

On June 20, 1997 the test structure came under flow attack for the first time. Some erosion had occurred in almost all sections along the edge of the falling apron already before June 24, 1997. Further observations of the development of the falling apron in the different sections are summarized in Table 5.3-1. In addition to the visual inspection by the monitoring team photographs were taken regularly. A selection of these photographs was included in the Progress Reports.

Date	Section	Material	Observation
24 to 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 to 26/06	C	geo-sand containers type E	erosion over 2 to 7 m
26/06 and 02/07	C	geo-sand containers type E	bed scour
01 to 02/07	C	geo-sand containers type E	some erosion
08 to 09/07	C	geo-sand containers type E	erosion occurs at slope
10 to 11/07	C	geo-sand containers type E	erosion at river bed
15 to 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 fallen up to 5 m from the edge
before 24/06	D	0.40 m cc-blocks	blocks have fallen up to 1.5 m from edge

Table 5.3-1: Observation of the development of the falling aprons during the 1997 flood

Date	Section	Material	Observation
24 to 26/06	D	0.45 m cc-blocks	erosion over 5 m
24 to 26/06 26/06 & 02/07	D D	0.40 m cc-blocks cc-blocks	erosion over 1.5 m bed scour
03 to 04/07	D	cc-blocks	erosion occurs along slope
05/07	D	cc-blocks	cc-blocks fallen over slope
08 to 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 to 26/06	E	geo-sand containers type E	erosion over 7 m
24 to 26/06	E	0.45 m cc-blocks	erosion over 7 m
01 to 02/07	E	cc-blocks	bed scour
03 to 04/07	E	cc-blocks	erosion
05/07	E	cc-blocks	cc-blocks fallen over slope
08 to 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 to 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 to 26/06	F	gabion sacks	erosion over 5 m
(3, 4, 6, 11, 12, 15, 16 to 17)/07	F	gabion sacks	erosion
07/07	F	gabion sacks	scour
13 to 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 to 26/06	G	0.45 m cc-blocks	erosion over 4 m
(1 to 10, 12 to 14, 15 to 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 as measured from survey boat (see Annex 4)

Table 5.3-1 (continued): Observation of the development of the falling aprons during the 1997 flood

Standard cross-sections have been selected in the middle of each section (see Fig. 4.3-2) to describe the under-water bathymetry. These cross-sections have been surveyed almost daily, from the actual bankline about 200 m in the channel. For the comparison of the cross-sections measured at different times, the accuracy of the positioning system causes a limitation. The standard deviation of the positioning by DGPS was between 2.1 m (Astech) and 4 m (Trimble). The electronic distance measurement (EDM) by using total station of the manufacturer Wild had an accuracy of ± (2 mm + 2 ppm).

5.3.2 Slopes

The interaction between the slope of a riverbank and the occurrence of slides and slips is known. Phenomena of the formation of steep cliffs along banks have been studied already by Spring (1903). He observed the formation of steep sub-surface sand cliffs along banks and realised that a vertical cliff weakens the protection of the embankment.

The thickness of the falling apron should be sufficient to cope with these cliffs. According to the field experience by Spring (1903), it is possible to prevent the development of such a cliff by an apron of basalt stones with 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 can be eroded within a few hours.

Varma et al (1989) mention that observations at guide banks on various rivers have shown actual slopes of launched aprons ranging from 1V:1.5H to 1V:3H and in some cases even flatter. However, in most cases the average approximates 1V:2H. Varma et al stated: "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 river bed and are washed away."

Natural slopes of a free eroding bank, which is not influenced by a nearby structure, can be estimated by the observations during the erosion of the earthen dyke left after construction in front of the falling apron. In 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.

The average **slope of the falling aprons** was 1V:2.0 H in Sections C to H-1 after they had started to function in the first two test months of 1997.

The average slope confirms the observations by Varma et al (1989) and corresponds to the design. This relatively steep slope might be considered as a factor contributing to the risk of small slides or slips, which were observed in these sections. In the following, special remarks are given to Sections C and H, which also confirm the observations in the literature:

The slopes of the falling apron in those Sections C and H have been surveyed in June, August and September 1997:

upper part:	average slope: 1 V:1.0 H	steepest slope: 1 V: 0.46 H
lower part:	average slope: 1 V:1.6 H	steepest slope: 1 V: 1.2 H

(a) Section C

The geo-sand-containers of the falling apron in Section C did not fall gently, 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, but the side scan sonar survey indicated that the coverage was very irregular.

(b) Section H

From photographs it can be seen that the boulders of the falling apron in Section H-1 had fallen and that the geotextile BIDIM S 390 was hanging on a steep slope of up to 1V:0.46H. After the flood season, the falling apron had been eroded over a distance of about 25 m. This erosion started when a scour hole had developed near the downstream termination (see Fig. 5.3-1).

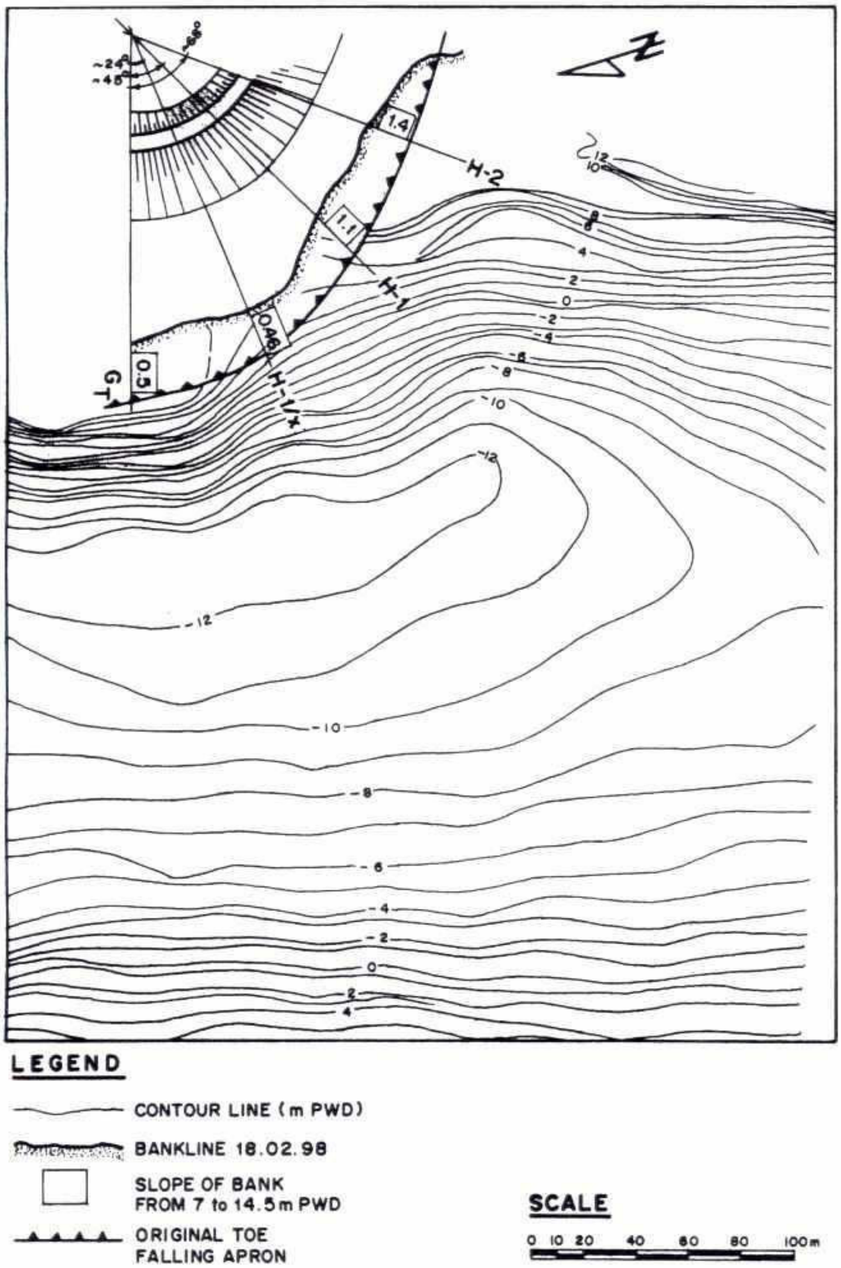


Fig. 5.3-1: Steep slopes in front of the falling apron in Section H

In 1998 the slope of the falling apron varied between 1V:1.3H and 1V:2.0H. This is comparable with the slopes measured during the previous monsoon. The steepest slope 1V:1.3H had been surveyed in Sections F to H-2 end May. Later in the monsoon the slopes became less steep.

The steep bluffs or cliffs, which occurred in Sections C and H in 1997, did not develop in 1998. It might be a general tendency that steep bluffs become gradually gentler when the scour hole in the second year is stable or less deep than in the first year. However, the maximum slopes observed in the structure were steeper than the value of 1V:2H assumed in the design rules.

The regular falling process has been disturbed by two **small slides** in Sections C and H (see Table 5.3-2). In the period of the second slide from August 28 to 30 1997, 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. No slides or slips of the falling apron have been observed during the 1998 monsoon.

Material of apron	Cross-section No.	survey before slide date	survey after slide date	visual observation of erosion dates	minimum water level m+PWD
Geo-sand container	C	June 11	June 20	beginning July	10
Rip-rap	H-1	August 20	August 27	August 28-29	9

Table 5.3-2: Two small slides in the falling apron process in 1997

The steepest slopes in the upper part of the cross-section H-1 were observed close to the scour hole (see Fig. 5.3-2), where the cross-sections G and H-1/x intersect the deepest part of the scour hole. The upper part of the scour hole had a depth of 4.5 to 5.5 m from 9 m+PWD to the excavation level of the falling apron at 14.5 m+PWD.

The 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 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 underwater slope of the test structure. In general, repair measures are necessary to maintain a good protection of this underwater slope. In the winter season 1997/98 such a repair measure was carried out in the area of the downstream termination, where the deep scour hole was close to this falling apron.

The subsoil investigation during the Study Phase of the Project included boring BA-1 in the Bahadurabad area (see Annex 13 of the Final Report Planning Study Phase, Consulting Consortium FAP 21/22, June 1993 and also Fig. 5.3-2). 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. 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.

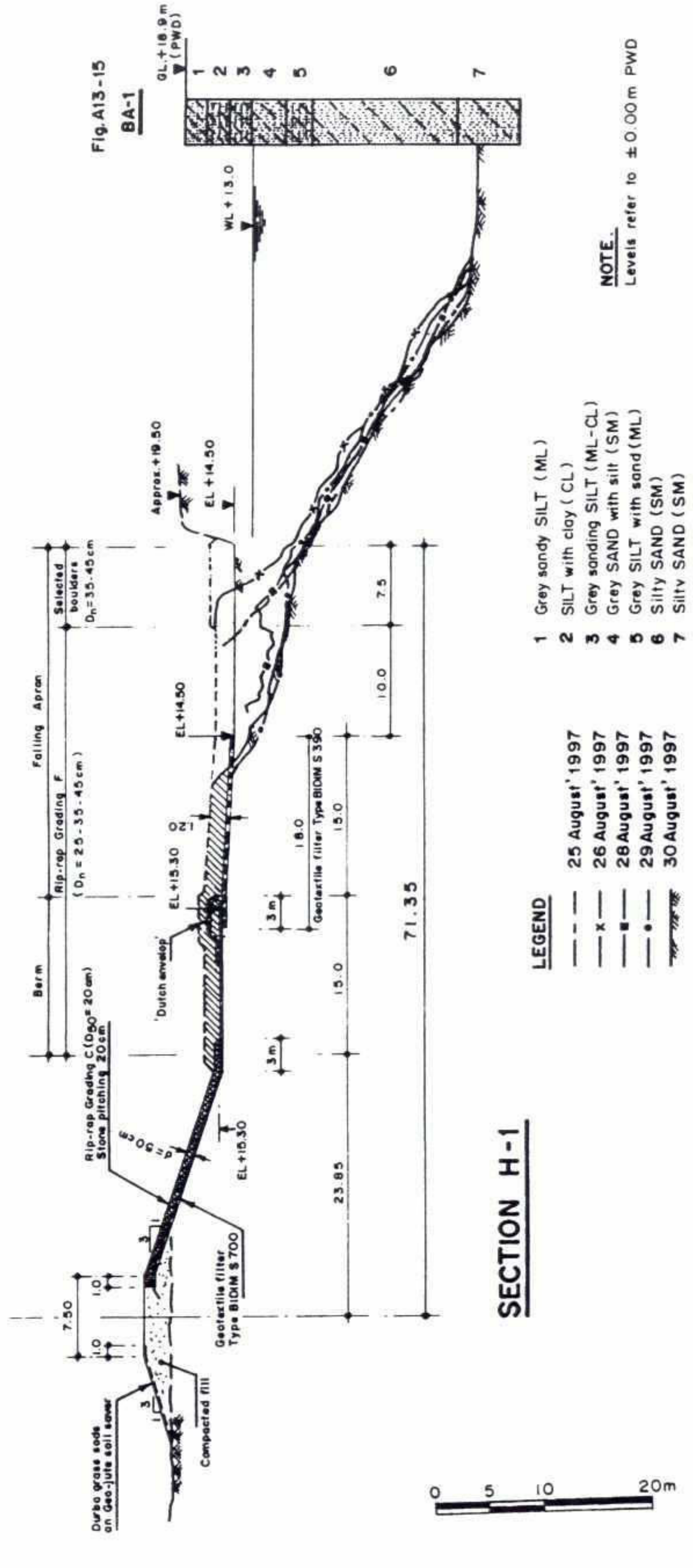


Fig. 5.3-2: Cross-section H-1 with falling apron

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

$$\tau = c + \sigma \tan \varphi \tag{5.3-1}$$

in which

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

The characteristic values of these parameters in Bahadurabad are given in Table 5.3-3. 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 et al, 1993). This means that falling aprons with a slope of 1V:0.5H in a toplayer and 1V:1.6H in sublayers can be expected in general.

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

Table 5.3-3: Soil parameters in Bahadurabad

Finally, the following conclusion were drawn. 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 with an average of 1V:1.6 H occur. 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.

The subsoil investigation in Bahadurabad shows that the 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 the toplayer and the slides are usually limited in size. However, the risk of larger slides is unknown.

5.3.3 Erosion Rates

Through comparison of bathymetric cross-section surveys average erosion or sedimentation rates can be calculated. They are calculated in horizontal direction, as this erosion direction is critical for the stability of the revetment.

The falling process progressed very gradually as shown exemplarily for Section F in Table 5.3-4, where the average erosion and sedimentation rates for certain period during monsoon 1997 in relation to the distance from the revetment crest are given. The erosion rate started with about 0.2 m/day in June/July 1997 and 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 1997, 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 the parallel flow attack, with velocities not higher than 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 University of Bremen.

Selected surveys in 1997			Distance from crest of revetment (m)		
start	end	days	70 - 80	80 - 90	90 - 100
June 26	July 05	09	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 5.3-4: Average erosion/sedimentation rates [m/day] in Section F in 1997 monsoon

The conclusion is that the facing slope of the falling apron in Section F had continued to erode at an average velocity of 0.10 m/day as long as the channel eroded. The erosion only stopped in the 1997 monsoon due to the general sedimentation in the channel.

From June 1998 onwards the channel in front of the revetment continued to silt up. The facing slope of the falling apron silted also up, but less than the sedimentation in the channel. After August the channel started to erode and the facing slope eroded also. The erosion velocities ranged from 0.01 to 0.07 m/day. The facing slope had eroded only several meters during the 1998 monsoon. Most erosion occurred in Section E-2 and H-1. In Section H-2 sedimentation continued up to the end of August. In September and October some erosion took place in that section. In comparison with the 1997 monsoon the facing slope had been relatively stable in 1998.

This means that the sloping face will probably start to erode as soon as the channel erodes again. Consequently, the sloping face has not reached an equilibrium position during the moderate attack by a parallel flow. This might be because the cc-blocks and the gabion sacks do not form a filter layer. The facing slope could be stabilised by dumping a layer of graded material, which will turn into a filter layer by natural grain sorting.

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 occurs during small slides, which take one or two days to develop. Near the deep scour hole the maximum erosion velocity has been measured at Section H-1: 1.5 m/day. No monitoring data are available to assess the maximum erosion velocity in the slide in Section C.

5.3.4 Coverage of Falling Apron

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 (Consulting Consortium FAP 21/22, 1998).

A kink in the depth measured in cross-section surveys indicates a certain coverage down to this kink, (see example in Fig. 5.3-3). This is caused by the slowly eroding surface formed by the falling elements. Also Spring (1903) mentions this phenomenon from field observations on the large rivers in the Indian sub-continent (Fig. 5.3-4).

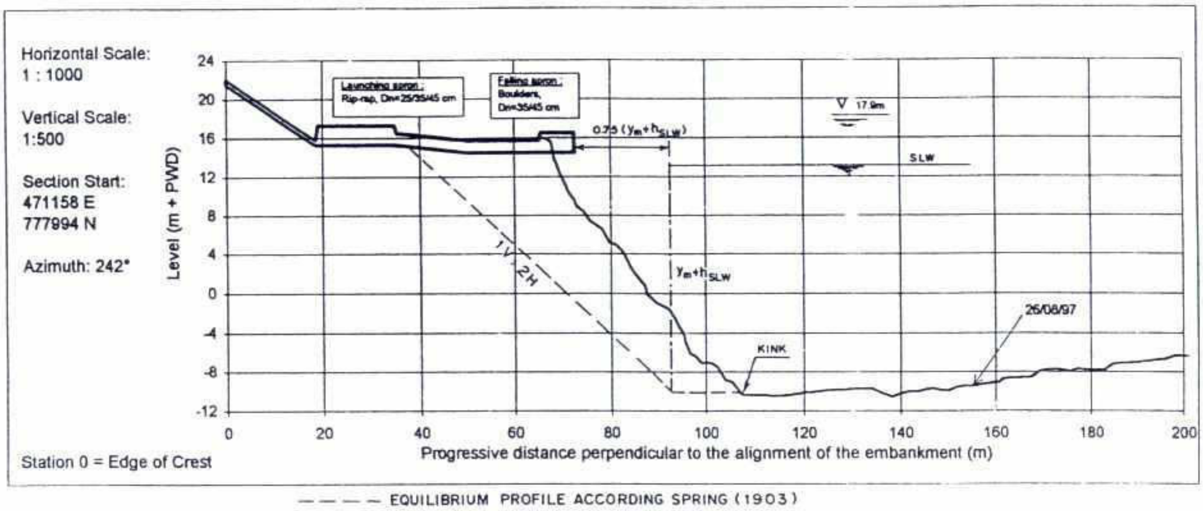


Fig. 5.3-3: Cross-section H-1 on August 26, 1997

The results of the physical model investigations in France showed 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 by only changing the block size.

Nevertheless during the successive monsoon floods the structure stability was not endangered and the slope of the falling apron remained relatively stable in position.

292

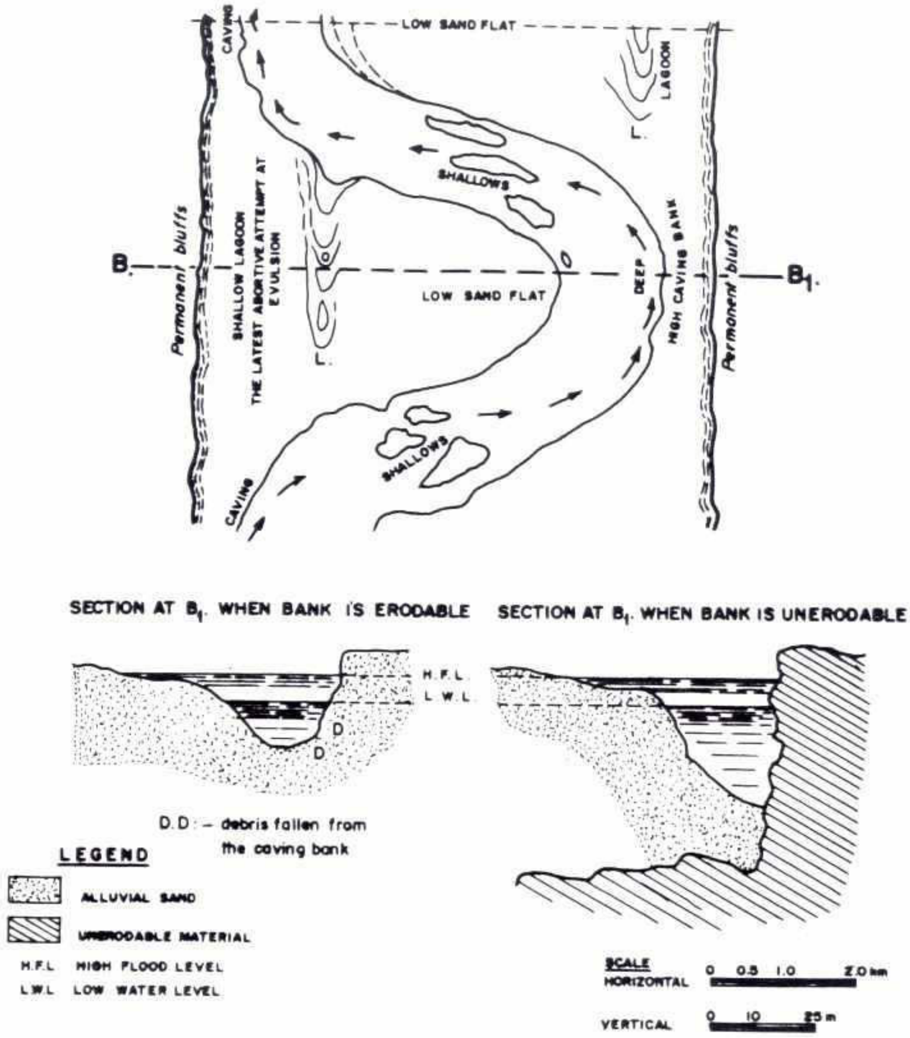


Fig. 5.3-4: Action at a river bend according to Spring (1903)

96

6 RIVER RESPONSE

6.1 CHANGES UPSTREAM AND DOWNSTREAM FROM THE TEST STRUCTURE

The bankline in front of the test structure started to erode in spring 1997 when the water level started to rise. During the first half of July strong bank erosion had been observed downstream from Section H (see Fig. 6.1-1). An embayment developed during the monsoon more or less as assumed in the design of the test structure.

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/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. The banklines in Fig. 6.1-1 show that the floods of 1997 and 1998 had eroded about 20 to 200 m/year of floodplain area.

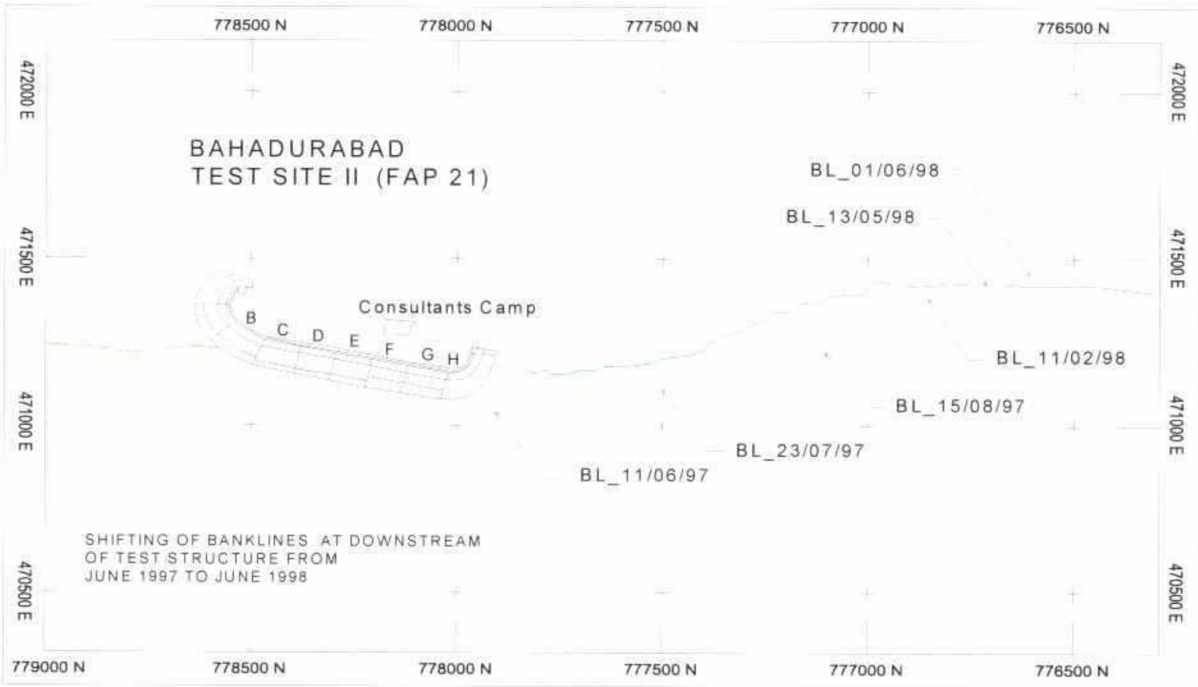


Fig. 6.1-1: Bank erosion from June 1997 to June 1998

Upstream from the revetment structure no bank erosion occurred during the monsoon 1997 and the bankline was almost stable over a distance of 600 m during the 1998 monsoon. However, further upstream some bank erosion had been observed in 1998. In the design an upstream bank erosion of 100 m had been anticipated, which did not occur. If the upstream bank erosion would have been more than 100 m, then the falling apron of the upstream termination would probably need reinforcement due to the formation of a scour hole. Basically, no structure-induced changes can be identified upstream from the test structure.

At the end of the 1997 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. In the period January to

June 1998 the protrusion of the downstream edge of the falling apron had slightly increased from 80 to 110 m. The protrusion of the crest of the revetment had increased from 20 to 30 m. After June the bank line (defined as the edge of the floodplain) upstream and downstream from the test structure became stable.

Given the limited predictability of the distances of bank retreat, it is impossible to assess whether the bank in this area would have been eroded less or more in a situation without the Revetment Test Structure. This is illustrated in Fig 6.1-2.

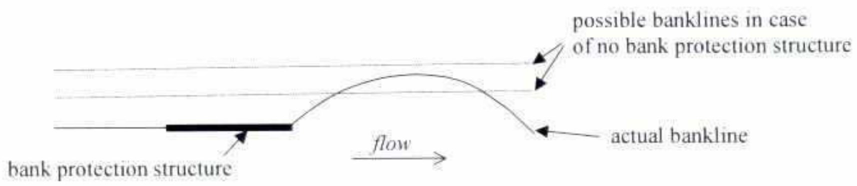


Fig. 6.1-2: Impossibility to assess net effect of impeded bank erosion at the test site and embayment formation immediately downstream

Despite the impossibility to assess the net total effect of the Revetment Test Structure, one can argue that the presence of the structure enhances bank erosion in the area immediately downstream in two ways. Firstly, the structure-induced local scour decreases bank stability through oversteepening. Secondly, flow attraction by the structure-induced scour hole leads to a more perpendicular flow impingement on the bank. These processes were active during the dry season 1997/98, but the attack ceased already considerably during the subsequent flood of 1998 when the main channel moved away from the structure.

6.2 CHANGES IN THE TEST SITE AREA

The primary effect of the revetment has been the effective stopping of bank erosion at the very site of the test structure. At the same time the structure has produced bed scour, which was most pronounced during the flood of 1997. Moreover, a channel confluence was formed downstream from the structure during the dry season 1997/98. This was a result of flow attraction by the structure-induced scour hole (Subsection 6.2.4 of Annex 1). The presence of a confluence produced additional local scour. The scouring in front of the test structure is described in Section 7.

290

7 SCOURING

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 and one near the downstream termination of the revetment structure due to strong turbulence in a vortex street associated with flow separation between an eddy and the main flow. This eddy caused a deep scour hole and bank erosion downstream from the test structure over a length of 100 to 200 m. However, upstream from the structure bank erosion was very small during the 1997 and 1998 floods and therefore an excessive scour hole near the upstream termination did not develop. During the monsoon floods the revetment structure became more exposed to flow attack due to bank erosion downstream (Fig. 6.2-1). This resulted in coalescing scour holes into one scour hole due to protrusion scour and flow separation at the end of the 1997 monsoon. However, during the receding limb the scour holes tended to develop again separately. The scour development in 1998 had been dominated by rapid sedimentation at the onset of the monsoon and fast erosion after the highest peak flow of that monsoon (Fig. 4.3-8 to Fig. 4.3-10).

The monitoring data of this scour hole development was analysed. The most relevant aspects are described in the following sections. The maximum scour depth is treated in Section 7.2, its location in Section 7.3, the slopes and the volume in Section 7.4. The risk of slides is discussed in Section 7.5 and the formation of a confluence scour in Section 7.6. Finally, a comparison with physical model investigation is carried out in Section 7.7.

7.2 LOCATION OF THE MAXIMUM SCOUR DEPTH

The location of the deepest point of the scour hole has been determined from bathymetric maps. In some surveys the scour hole had an elongated flat bottom, which is indicated by two or more points connected by a line in Fig. 7.2-1. In general, this deepest point moved in downstream direction during the 1997 and 1998 monsoon floods. In the following this will be analysed more in detail.

During June and July 1997 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 1997 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 and in October 1997 it started to silt up. The location of the deepest point did not move anymore.

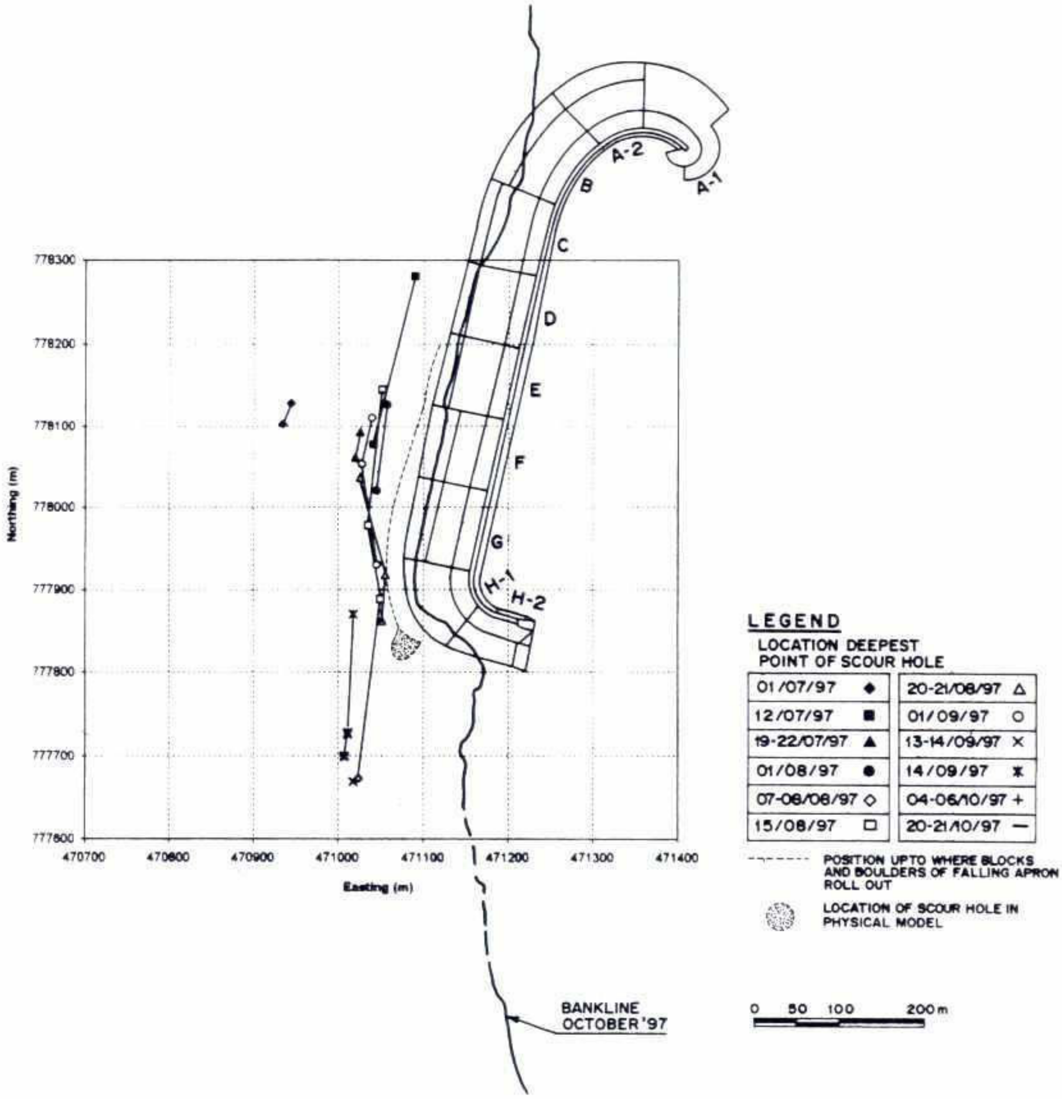


Fig. 7.2-1: Location of the deepest point in the scour hole 1997

7.3 MAXIMUM SCOUR DEPTHS

7.3.1 Definitions

The scour depth is defined as the elevation difference 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 riverbed points of a cross-sectional area, which is about 30 to 40 % of the total channel cross-sectional area. Due to these definitions and the nature of the physical processes the scour depths can only be analysed in combination with the bed level.

7.3.2 Observed Scour Depths and Erosion Rates

The channel bed level was almost constant during the 1997 monsoon. It varied only from -2.5 to +2.5 m+PWD. The bed levels and the scour depths have been surveyed during the monsoons 1997 and 1998. The data of 1997 is presented in Fig. 7.3-1. The bed in the approach channel remained constant at 2 m+PWD until June 1998.

The depth of the scour hole in front of the revetment structure gradually increased from 2 m in May 1997 to 14 m at the end of September 1997 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 Section 5.3). The gradually increasing scour depth is different from the quick development of the local scour holes near the Kamarjani groynes in the 1995 monsoon (erosion rates of 0.3 m/day). The slow erosion rate of the scour hole near the revetment structure has possibly reduced considerably the risk of sudden large slides.

After the monsoon flood in 1997 a deep scour hole remained at the downstream side of the revetment with its centre near Section G (see Fig. 7.3-2 to 7.3-7). The scour depth remained almost constant at 10 to 11 m during the first months of 1998 until June (see Table 7.3-1). The depth of the second scour near the downstream termination was 8 to 9 m in the same period. The main scour hole with its centre near Section G extended to the downstream termination scour hole to form one big hole.

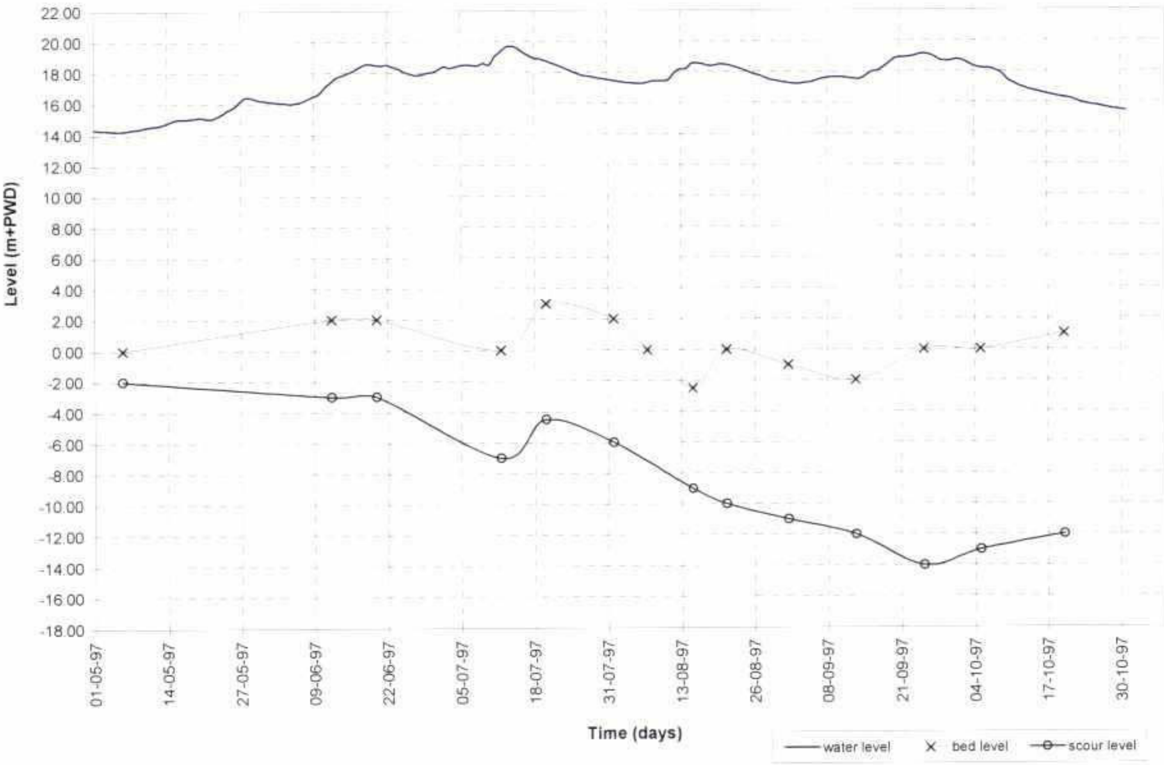


Fig. 7.3-1: Development of bed level and scour depth in 1997

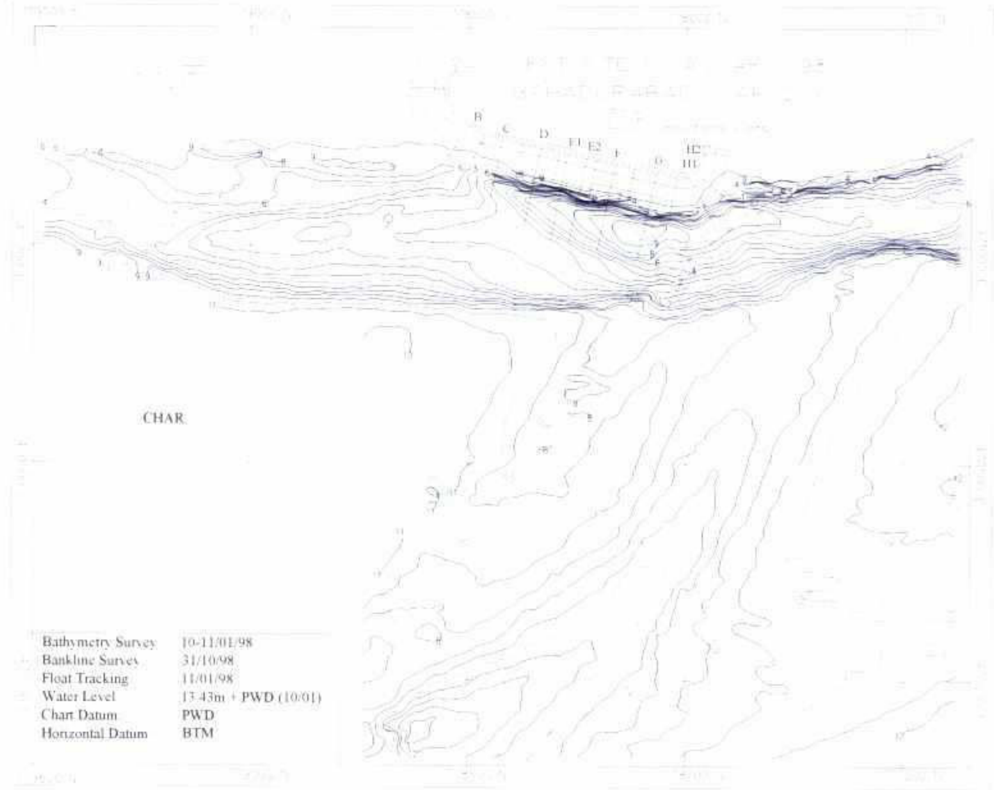


Fig. 7.3-2: Bathymetry on January 10/11, 1998

Fig. 7.3-3: Bathymetry on February 08, 1998

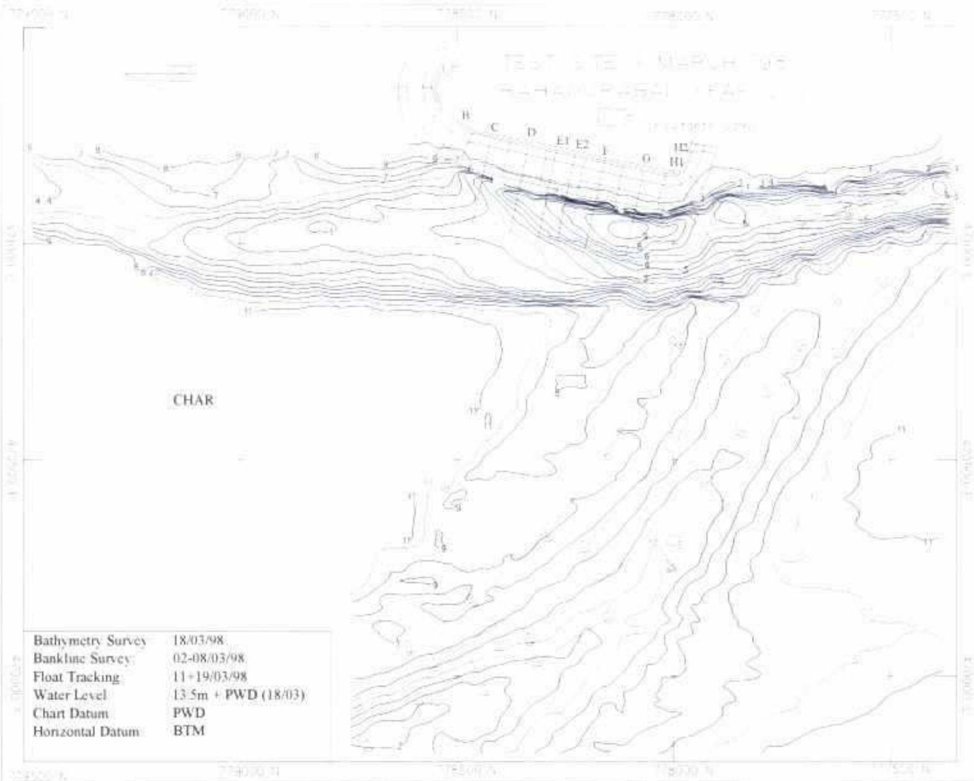


Fig. 7.3-4: Bathymetry on March 18, 1998

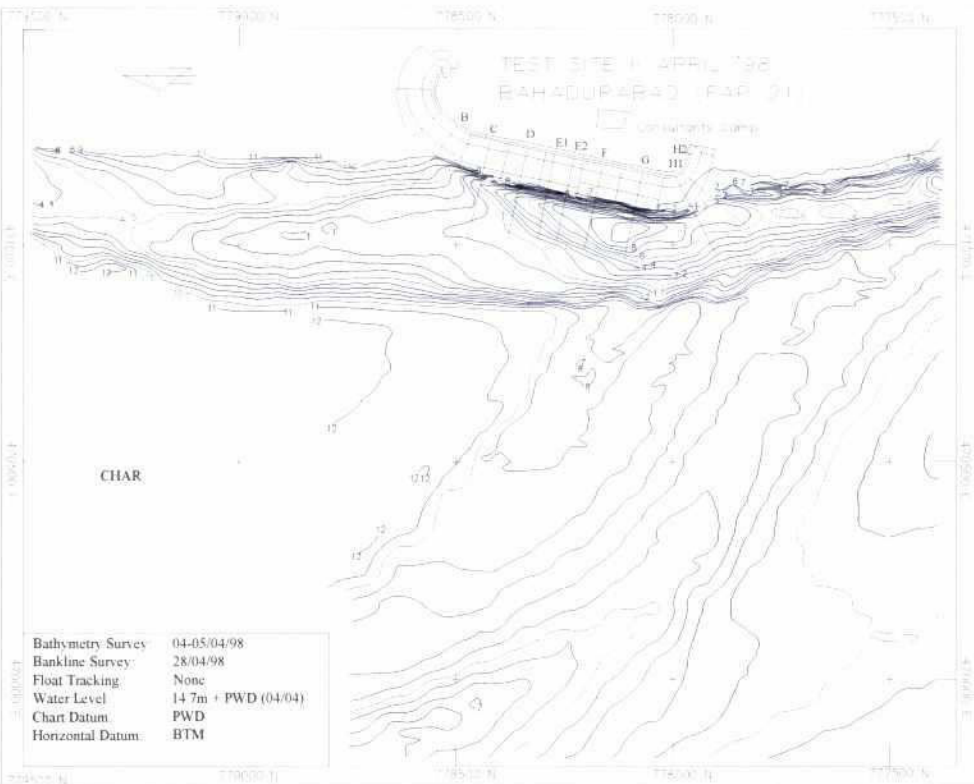


Fig. 7.3-5: Bathymetry on April 04/05, 1998

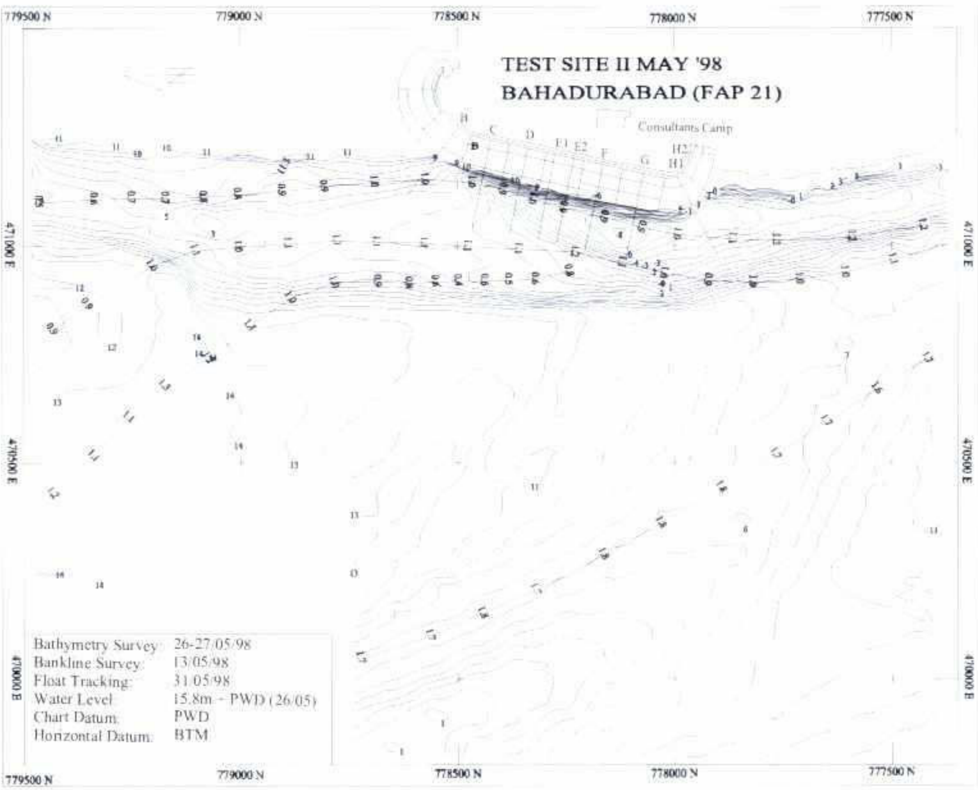


Fig. 7.3-6: Bathymetry on May 26/27, 1998

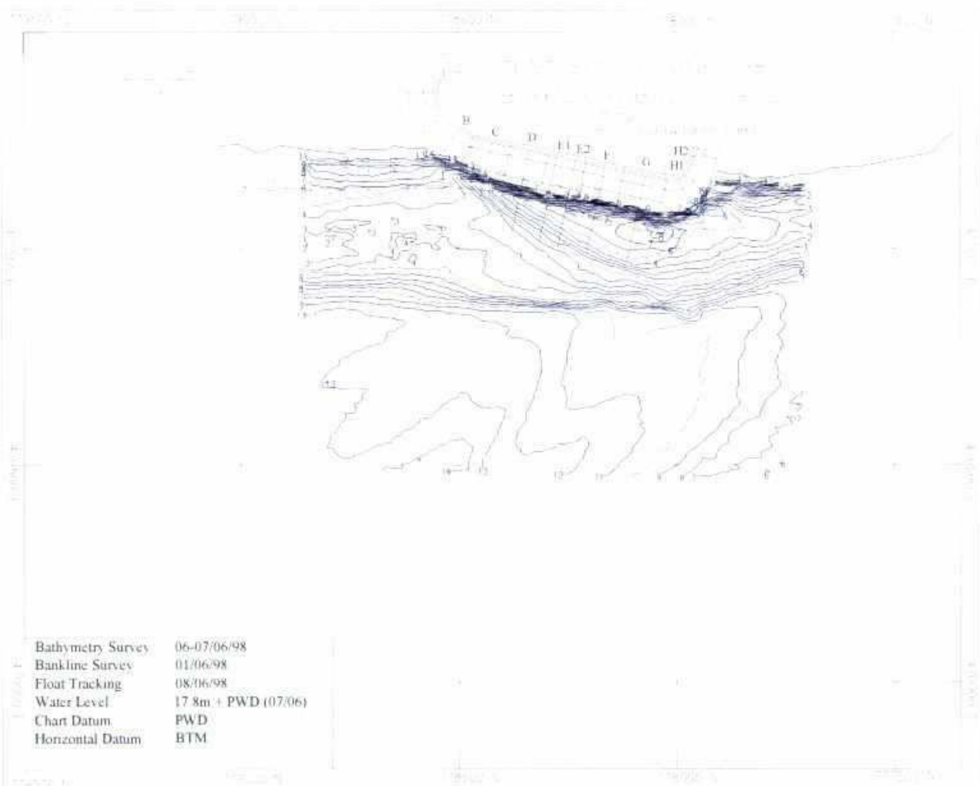


Fig. 7.3-7: Bathymetry on June 06/07, 1998

Between June 14 and July 11, 1998 fast sedimentation took place in front of the test structure, about 0.5 m/day (see Fig. 7.3-8 to 7.3-10). The bed level in the approach channel increased from 2 m+PWD to 15 to 16 m+PWD on August 09. The source of this sedimentation is some erosion of the bank upstream from the railway ghat and development and migration of a deep upstream scour hole.

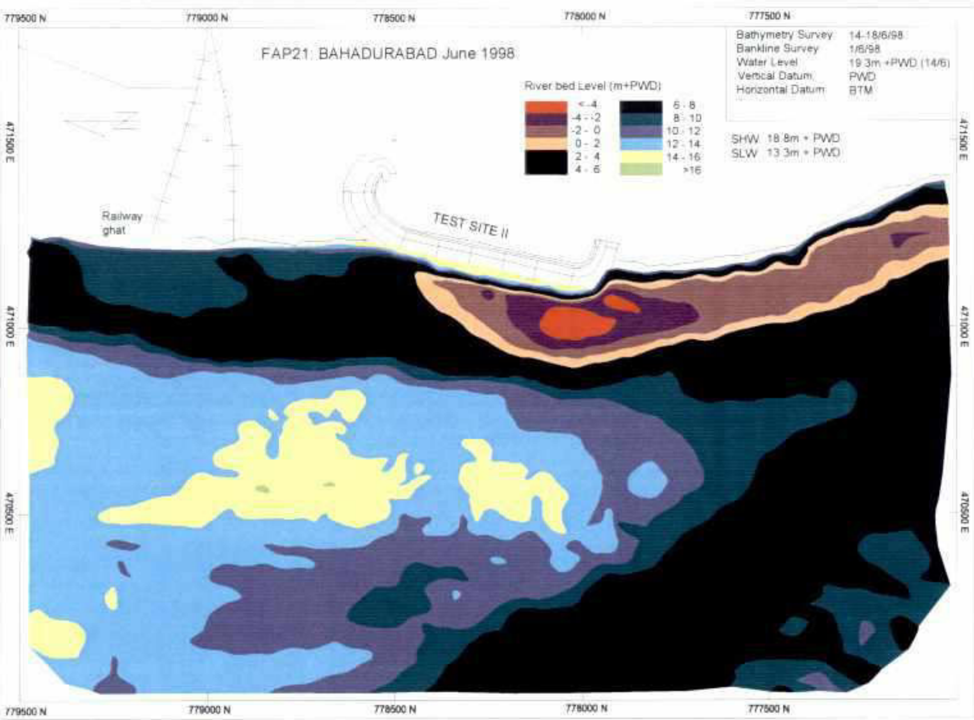


Fig. 7.3-8: Bathymetry on June 14 to 18, 1998

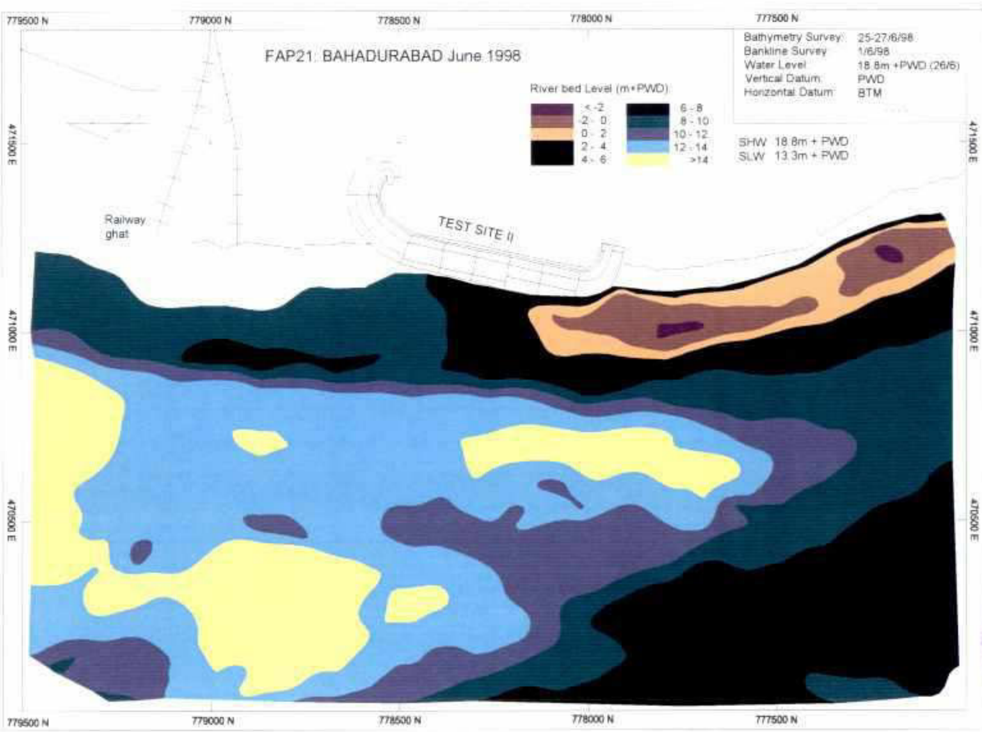


Fig. 7.3-9: Bathymetry on June 25 to 28, 1998

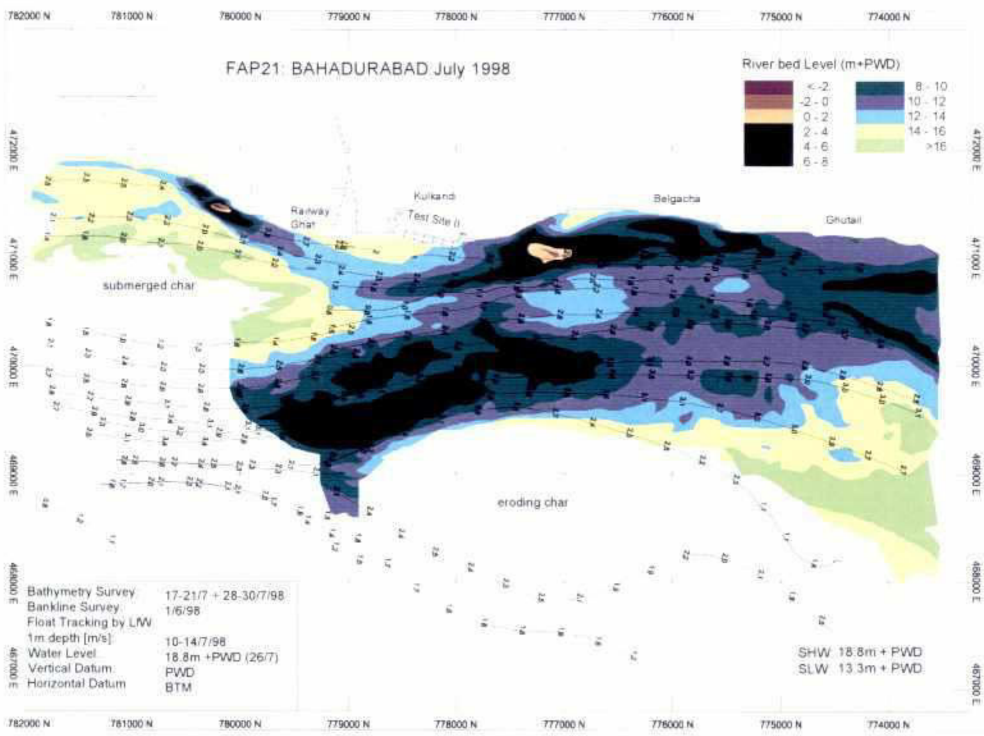


Fig. 7.3-10: Bathymetry on July 17 to 30, 1998

The riverbed in front of the revetment started to erode again after August 09, 1998. The erosion rate of 0.25 m/day in the channel from August 09 to September 10, 1998 was higher than in the year before. On September 10, during the highest flood peak, the river bed was very active with huge dunes of a height of 6 m to 8 and a length of 350 m (see contour lines in Fig. 7.3-11 and longitudinal sections in Fig. 7.3-12). In October the riverbed was flat again and it continued to erode to 3 to 4 m+PWD at the end of November (see Fig. 7.3-13 to 7.3-16).

survey date in 1998	bed level (m+PWD)	maximum scour depth (m)	near sections	max scour depth near downstream termination (m)	remarks
March 01 to 02	2	11/12	G	9	
April 04 to 05	2	12	G	10	
May 08 to 09	2	10	G	8	
May 26 to 27	3	11	G	11	
June 14	4/7	11	H	10	sedimentation
July 11	12/13	2	G/H	3	sedimentation
August 09	15/16	0	-	0	erosion
September 10	5/10	0	-	0	erosion, dunes
October 22	7/8	2	G	2	erosion
November 19	4/5	4	E	2	erosion
November 23	4	2	C	2.5	erosion
December 06	4	1	C/E	0	erosion
December 30	3	3	E	0	erosion

Table 7.3-1: Maximum scour depth near revetment in 1998

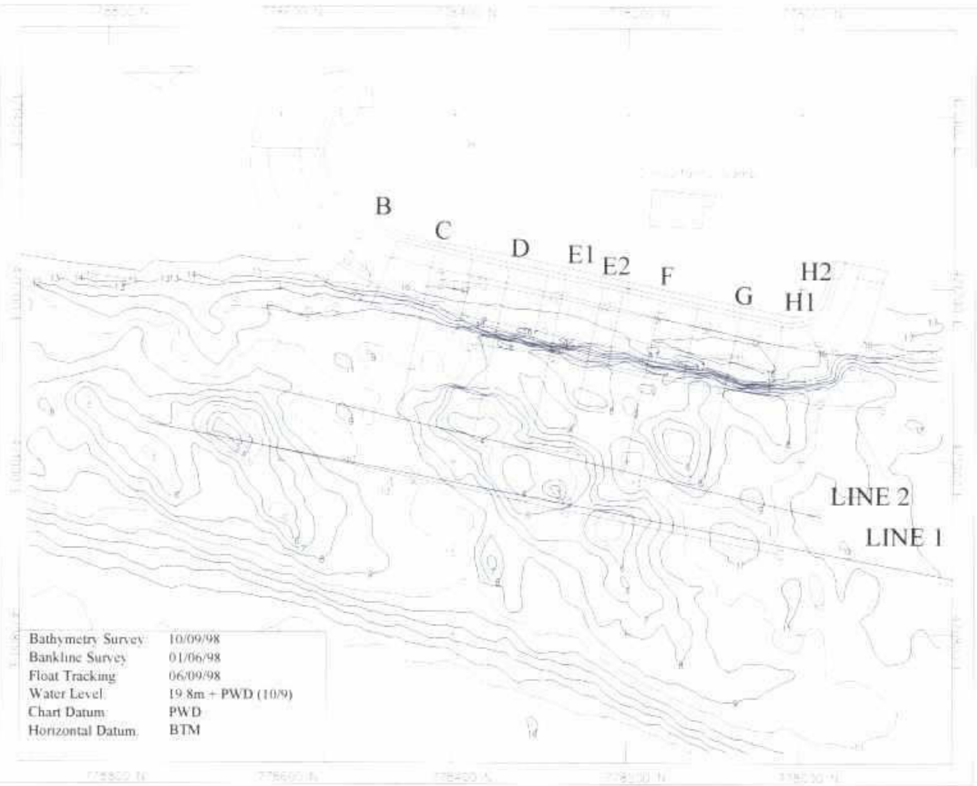
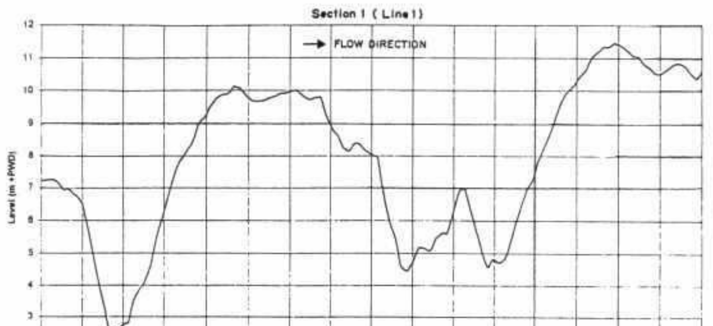


Fig. 7.3-11: **Bathymetry on September 10, 1998 with two section lines (the longitudinal sections, Fig. 7.3-12, are marked as lines 1 and 2)**



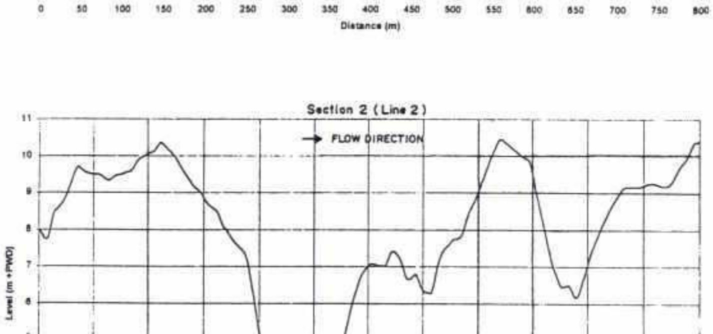


Fig. 7.3-12: **Dunes and bedforms measured on September 10, 1998**

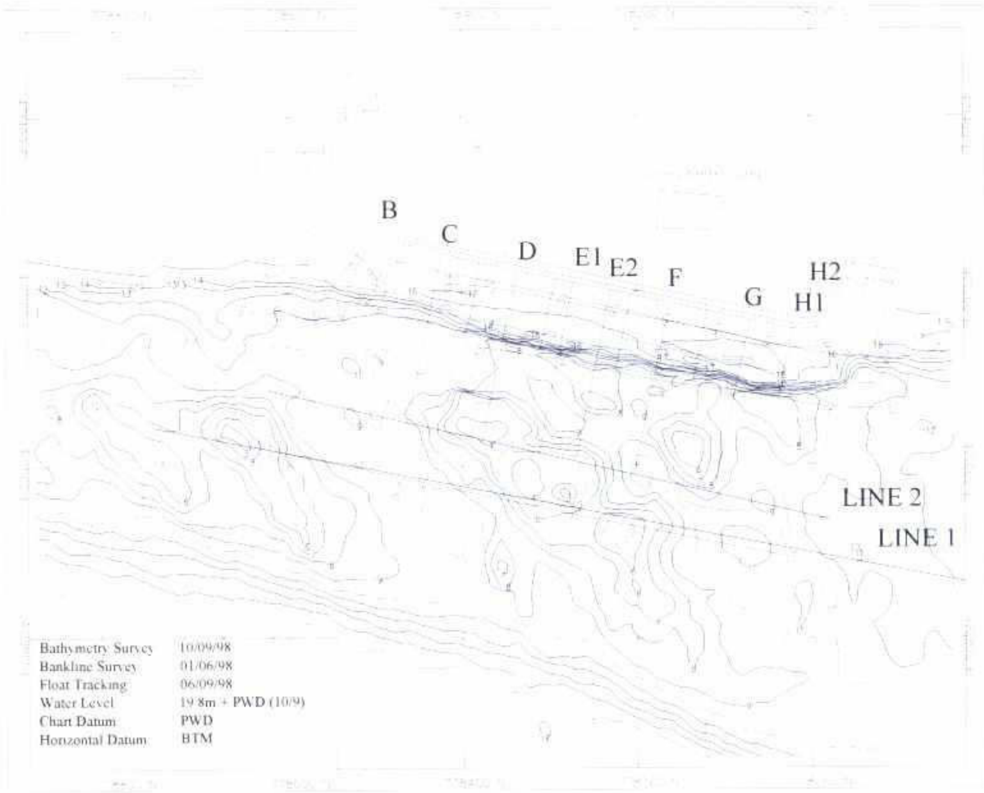


Fig. 7.3-13: Bathymetry on September 14 to 19, 1998

Fig. 7.3-14: Bathymetry on October 13 to 16, 1998

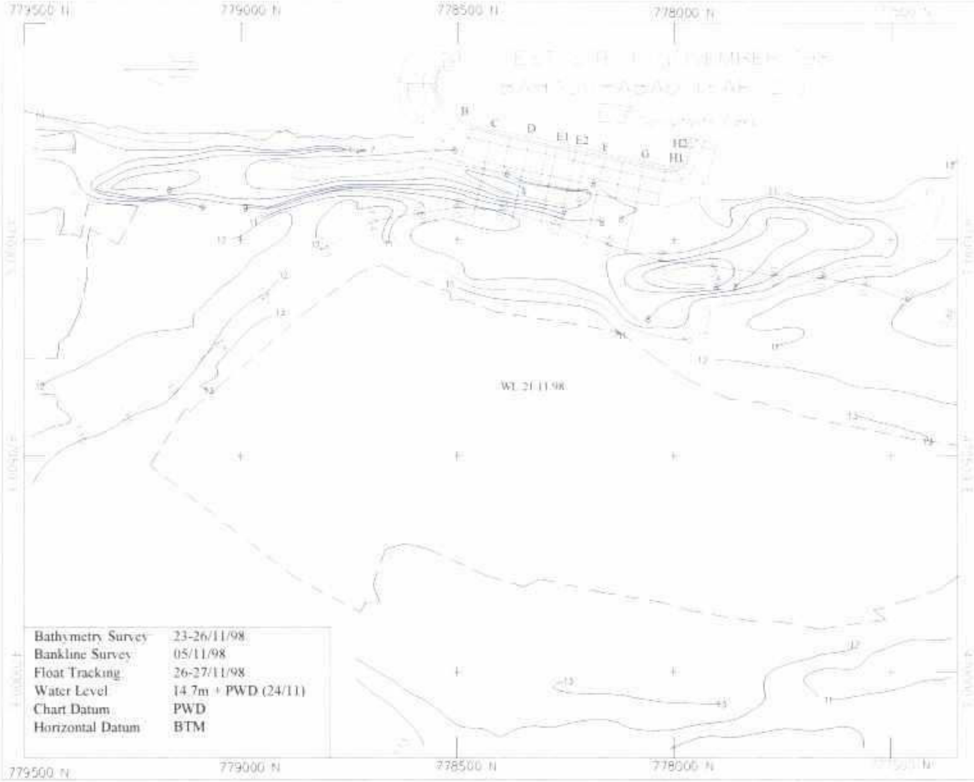


Fig. 7.3-15: Bathymetry on November 23 to 26, 1998

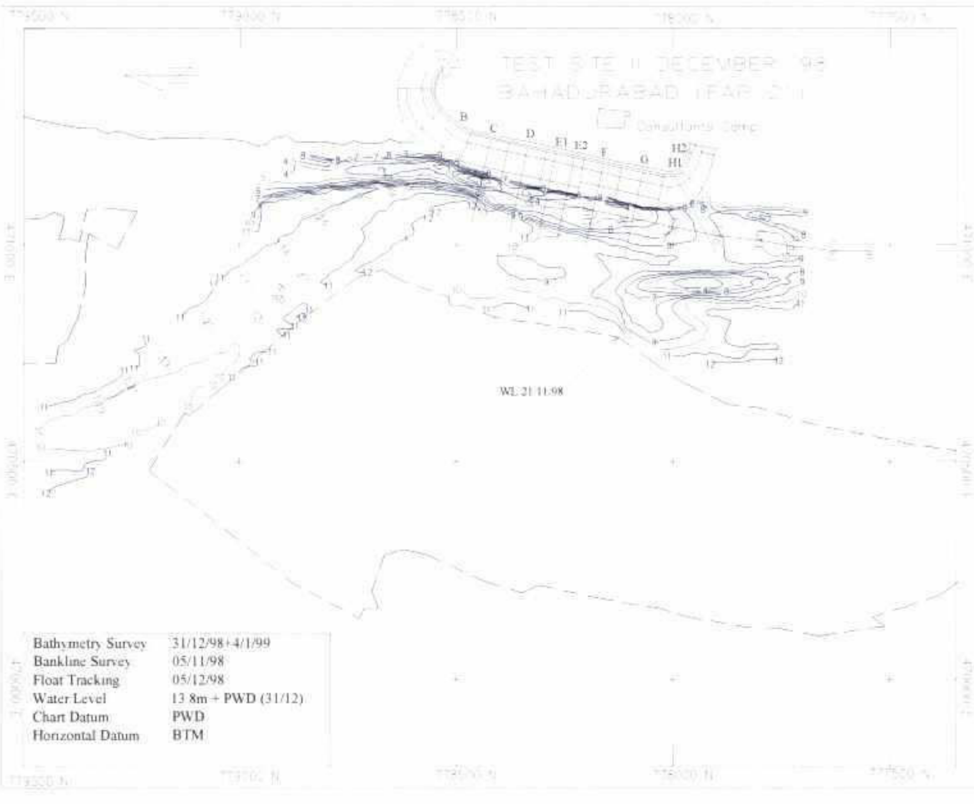


Fig. 7.3-16: Bathymetry on December 31, 1998

7.4 SCOUR SLOPES AND VOLUME

7.4.1 Slope in Longitudinal Direction

The longitudinal slope of a scour hole is the slope parallel to the main flow direction. The transverse or side slopes are almost perpendicular to the longitudinal slopes. The longitudinal slopes of the scour hole have been determined from three surveys in 1997 at the time the scour hole had been fully developed (see Fig. 7.4-1 to 7.4-4 with bathymetric maps where also the longitudinal alignment of the Sections A-A', B-B' and C-C' respectively is shown).

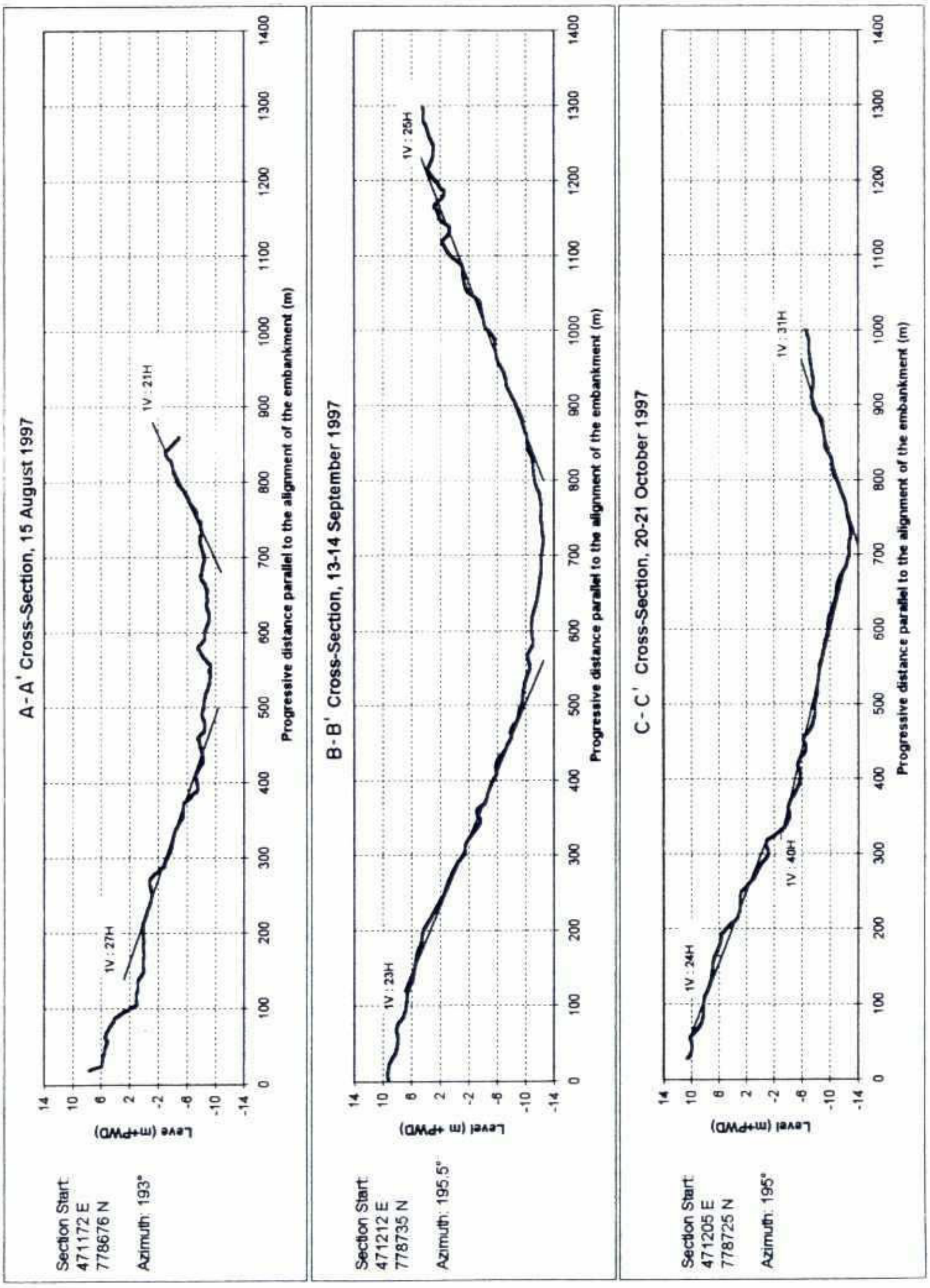


Fig. 7.4-1: Slopes in longitudinal sections of the scour hole

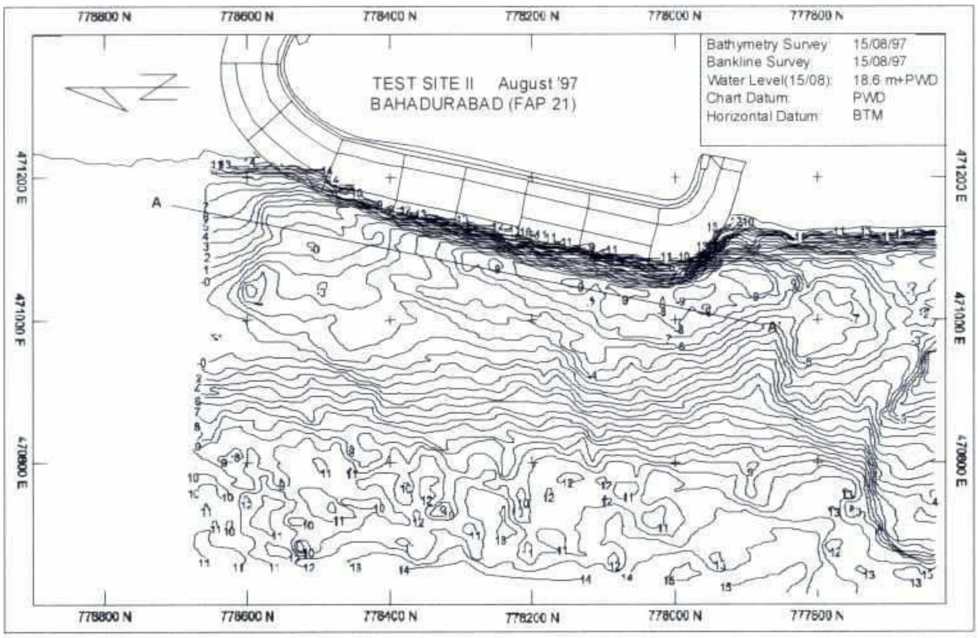


Fig. 7.4-2: Bathymetry in August 1997 with longitudinal Section A-A'

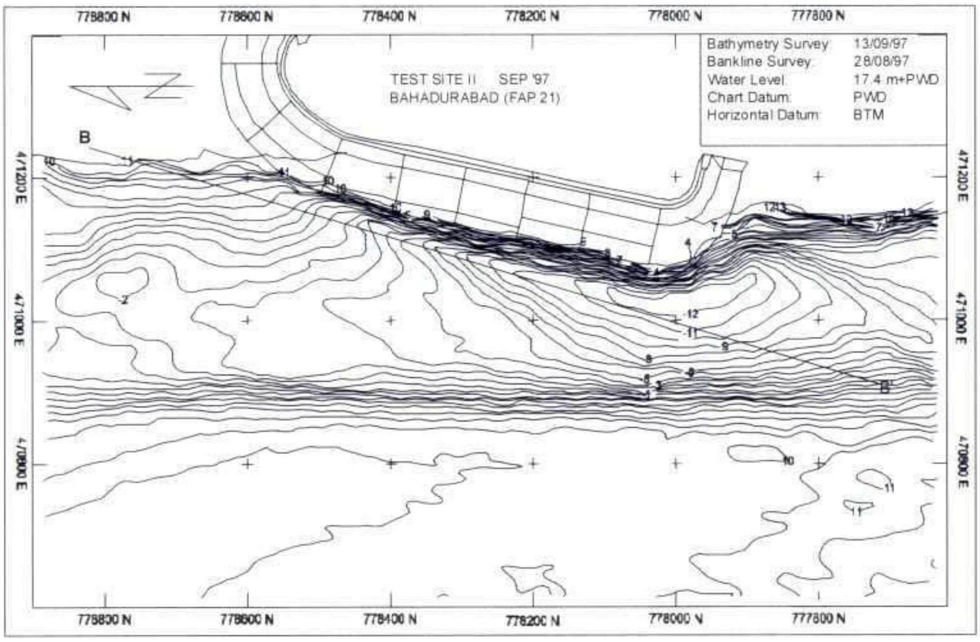


Fig. 7.4-3: Bathymetry in September 1997 with longitudinal Section B-B'

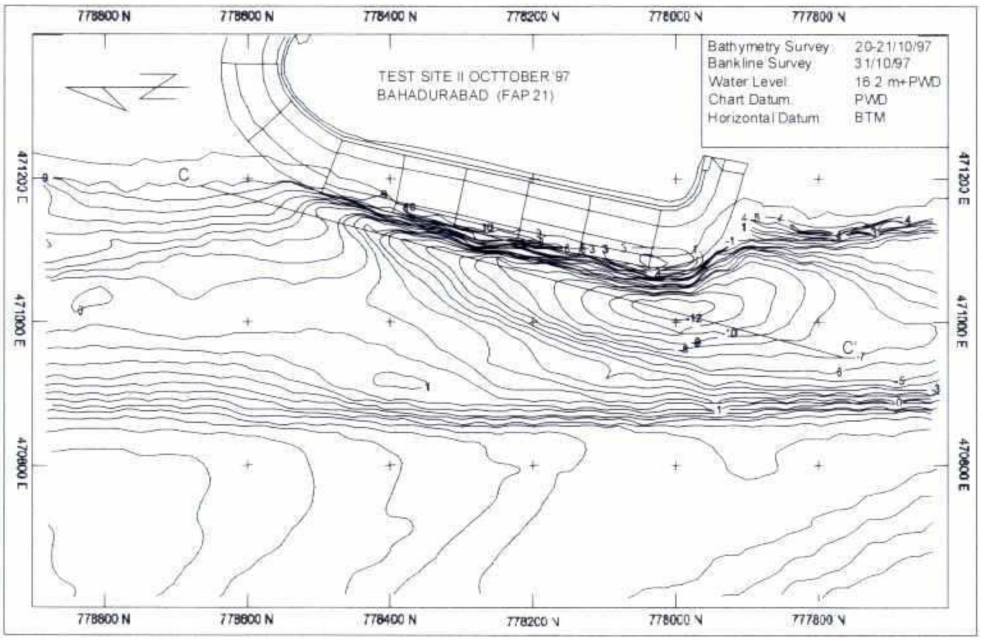


Fig. 7.4-4: Bathymetry in October 1997 with longitudinal Section C-C'

The fully developed scour hole is characterised by a long flat bottom with a length of 15 y_s, (y_s = maximum scour depth). Table 7.4-1 contains the average upstream and downstream slopes. With an average slope of 1V:25H, the maximum area A_l of a longitudinal section of such a scour hole is $A_l = 40\ y_s^2$. In October 1997 the scour hole started to silt up, its shape changed with sedimentation from upstream and the flat bottom had a slope of 1V:40H.

Date in 1997	Upstream slope (1V:xH)	Downstream slope (1V:xH)	Volume below -4 m+PWD (m^3)
August 15	27	21	173,000
September 13 to 14	23	25	393,000
October	24	31	379,000

Table 7.4-1: Longitudinal slopes and volume of the scour hole measured in 1997

7.4.2 Slopes in Transversal Direction

At the end of August 1997 the sloping face of the falling apron had moved to the deepest point of the scour hole. The left-hand side (riverbank) slope which has been covered by cc-blocks and boulders, was rather steep and could be determined between 1V:2H and 1V:3H. The right-hand side slope of the fully developed scour hole was much gentler, on average 1V:15H (see Table 7.4-2). Even gentler slopes occur during developing stages of the scour hole. The approximate maximum transverse area A_t of a cross-section through the scour hole perpendicular to the main flow direction is about $A_t = 10\ y_s^2$.

Hb

Section	date in 1997	left-hand side slope * (1V:xH)	right-hand side slope * (1V:xH)
H-1	August 30	1.5 to 2.3	12 to 19
F	August 29	7.5	11
D	September 12	6	15
C	July 07	-	25

* looking in downstream direction

Table 7.4-2: Side slopes of the scour hole measured in 1997

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 than the 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-hand side slope of the scour hole in front of the revetment structure (Delft Hydraulics & DHI, 1996).

7.4.3 Volume of Scour Holes

The volume of the scour hole has been calculated from three bathymetric surveys in 1997 (see Table 7.4-1). These volumes are the volume of the scour hole below -4 m+PWD. This is an arbitrary level just below the deepest part of the approach channel. 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).

7.5 RISK OF SLIDES

The monitoring at Kamarjani (see Annex 6) has shown that rapid scouring may trigger slides. However, the risk of slides seems much smaller at Bahadurabad. Inspection of the bank lines near Test Site II at Bahadurabad shows many small slumps in the bankline just south of the test site, but it is likely that the soil conditions in that area differ from the conditions at the test site itself. The subsoil south of the test site has been deposited recently and is hence much younger than the subsoil at the test site. Therefore, the risk of slides seems comparatively small at the test site. No failures in this regards have been reported during the monitoring period.

7.6 FORMATION OF CONFLUENCE SCOUR

From January to February 1998 the large scour hole with its centre near Section G attracted the flow from a channel, which split the char in front of the revetment. The flow lines formed an angle of 65 degrees with the crest of the revetment (see Fig. 7.3-2 and 7.3-3). This channel joined the channel along the revetment just downstream from the revetment. This confluence produced a scour hole, which was connected to the scour hole near Section G, but was a few metres shallower.

In March and April 1998 this situation did not change (see Fig. 7.3-4 and 7.3-5). The confluence scour extended slightly in downstream direction. The flow lines in the char splitting channel made still 60 to 70 degrees with the crest of the revetment.

In May 1998 this angle reduced to 40 to 50 degrees (see Fig. 7.3-6). The first peak flow had a maximum level of 19.3 m+PWD and it inundated the char more than 4 m. After this first peak flow in June this char splitting channel had disappeared and the confluence scour hole became separate from the revetment scour hole. In July the scour hole near the revetment had been filled up completely (see Fig. 7.3-10) and downstream from the revetment the remnant of the confluence scour migrated slowly in downstream direction as a single hole, surveyed on July 17 to 30, 1998. In September this scour hole had disappeared, because the whole channel had been filled up.

After the monsoon flood a new small char splitting channel developed more upstream and it joined the channel in front of the revetment just upstream from Section B (see Fig. 7.3-15). In November and December 1998 the flow lines made an angle of 50 to 60 degrees with the crest of the revetment (see Fig. 7.3-16). In November the confluence scour hole was just separated from the scour hole in front of the revetment, but in December they joined to form a long narrow single hole with the deepest point at 1 to 3 m+PWD. The discharge through this char splitting channel was small and the flow velocities varied from 1.1 to 1.25 m/s in these months. This flow velocity caused small bank erosion on the right bank of the char splitting channel.

The observed morphological development is very important for the design of the revetment structure. It shows that the combination of a local scour hole and a confluence scour hole occurs probably much more frequently than assumed in the design of the Revetment Test Structure. These char splitting channels develop after the monsoon and it might be the case that most of them disappear during the monsoon as observed in the 1998 monsoon. If the scour hole of the revetment is deep, then it has a strong tendency to attract small char splitting channels. This phenomenon increases the risk of a confluence scour hole near the revetment.

On the other hand large char splitting channels do in general not make large angles with the crest of the revetment. A smaller angle results in less deep confluence scour holes. It is possible that a small char splitting channel develops into a larger channel which will not be filled up during the next monsoon. Not much is known about the probability of such a development.

The risk that a medium or large char splitting channel joins the channel in front of the revetment is unknown. So also the risk of an unfavourable combination of a confluence scour hole and a structure-induced scour hole is unknown. Such a combination is expected to make the scour much deeper than the design scour. This aspect requires more in-depth investigation when designing permanent structures. It should be given attention in physical model investigations to determine the design scour depth.

7.7 COMPARISON WITH PHYSICAL MODEL INVESTIGATIONS

In the physical model investigation executed during the Planning Study Phase in 1992 (Consulting Consortium FAP 21/22, June 1993) the scour hole near the upstream termination had been studied. In an additional model investigation in 1993 (Consulting Consortium FAP 21/22, August 1994) the complete revetment had been tested including the scour hole near the downstream termination. The average bed level in the model was at 0.3 m+PWD (-13 m+SLW). This is a similar bed level as observed in the field, which varied mainly between +2 and -2 m+PWD in the approach channel.

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In the physical model the maximum scour depth near the downstream termination was 10 to 12 m with a 25 years return period (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 apron 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 why in the field about 4 m deeper scour was measured than in the model.

The conclusion is that the maximum scour depths at Bahadurabad Test Site can most probably be reproduced reasonably well in a physical model. However, the previous physical model investigation did not yield the most unfavourable flow attack by an approach channel showing an angle with the revetment structure. The design channel cross-section used for the model investigation had the same cross-section as observed in the field. The envelope curve method, applied to determine the design cross-section on basis of surveyed cross-sections, seems to provide good results.

In the physical model tests carried out 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 occurred about 25 to 50 m closer to the downstream termination than measured in the field (see Fig. 7.2-1).

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8 RECOMMENDATIONS FOR FUTURE DESIGN ASSUMPTIONS

The analysis of the structural performance of the test revetment at Bahadurabad has led to the modification and improvement of certain design components. In Annex 8 of the Project Evaluation Report structural deficiencies, observed during the monitoring period, are analysed and recommendations for future design are given. However, it can be summarised that the Executed Design, based on the assumptions made, withstood the occurring hydraulic loads and could stop the bank erosion in the protected area.

Design parameters must always be chosen with an appropriate safety margin to allow for compensation of inaccuracies in predicting the actual site conditions. It should be noted that the design parameters described hereinafter apply only to the test site at Bahadurabad.

8.1 HYDRAULIC DESIGN PARAMETERS

(a) Design High Water Level

The Design High Water Level (DHW) of 21.10 m+PWD which originates from a frequency analysis with a 25-year return period has not been reached in the observed years. A daily rise of 0.7 m/day and fall of 0.25 m/day was observed. No implications for the design are necessary.

(b) Design Flow Velocity

The initially chosen Design Flow Velocities for Bahadurabad Test Site were 3.5 m/s for the revetment (river-sided end of launching apron) and 3.85 m/s for the most river-sided end of the scour protection (falling apron). Above the falling aprons the maximum measured depth averaged flow velocity was 3.1 m/s. Above the launching aprons it was 2.1 m/s. The maximum measured depth averaged flow velocity above the deepest point of the scour hole was 2.7 m/s. All values are below the design flow velocities. No implications for the design are necessary.

The number of measurements for data processing was very limited, but the assumed flow velocity distribution in vertical direction as well as in transversal direction of the revetment slope could be confirmed in general.

(c) Design Wave Height

For the 25-year return period the Design Wave Height was assumed at 1.0 m. Despite the fact that in some cases higher waves were reported, these were not breaking on the structure due to mainly eastern wind directions. Nevertheless, it may be recommendable to carry out more comprehensive wave measurements at certain locations to allow for a sounder database in the future.

8.2 MORPHOLOGICAL DESIGN PARAMETERS

8.2.1 Design Scour Depth

Based on the results of the physical model investigations under assumption of a more or less parallel flow approach, the design scour depth has been estimated at values of 12 to 14 m+PWD near the upstream termination and 10 to 12 m+PWD near the downstream termination. A design scour depth of 6 m+PWD had been assumed between these two scour holes (see Fig. 8.3-1).

Due to the unexpected oblique flow approach, the main flow attack on the structure was shifted slightly towards downstream from the upstream termination.

Because of the uncertainties in predicting the exact course of the approach channel and because it might be subject to change in the successive monsoon periods, it was recommended to assume a constant scour depth (according to the maximum expected value) along the revetment structure.

The physical model tests performed demonstrated that the maximum scour depths can be reproduced reasonably well in a smaller scale, provided that the proper approach conditions are used. However, it has to be mentioned that the approach conditions change as the river responds to the structure. Due to the formation of a char splitting channel and the combination of a confluence scour and a structure induced scour, the scour depth can be much deeper than the design scour. As mentioned before, this aspects requires more investigation.

Nevertheless it can be stated, that the structure was resisting the actual impacts from flow and scour, still keeping additional safety margins for future operational phases.

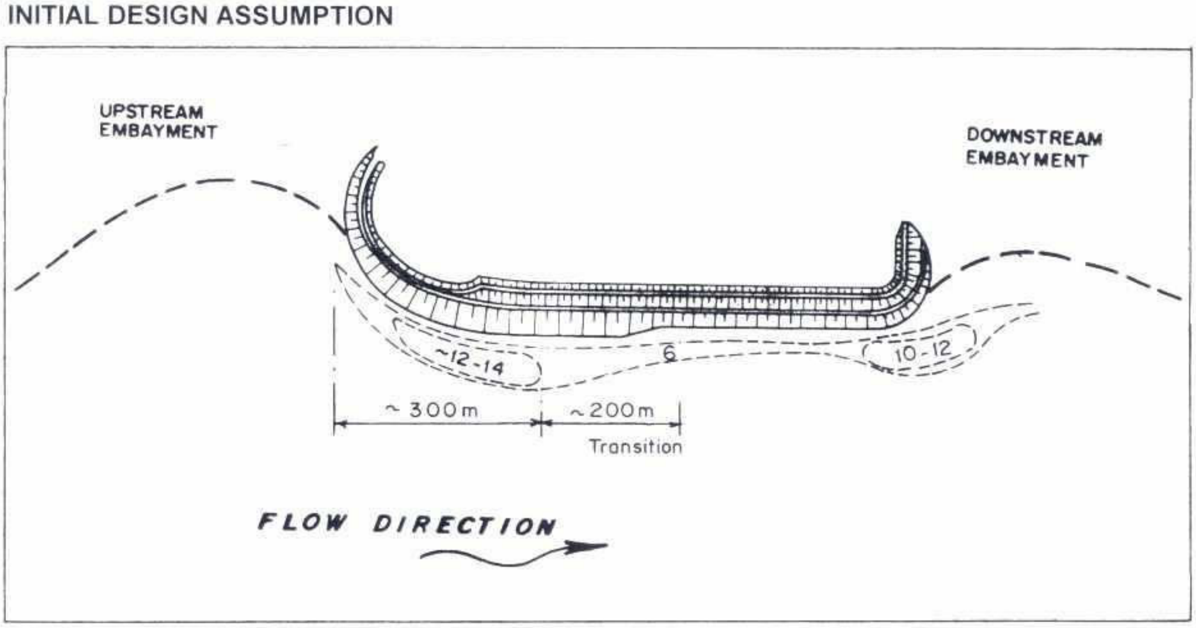

Fig. 8.3-1: Design scour pattern and bankline

8.2.2 Equilibrium Structure Induced Scour Depth

In general, structure induced scour is also affected by the reduced sediment supply due to the locally protected riverbank. This may lead to deeper scour because usually eroded material reduces the formation of near-bank scours. In Annex 7 this phenomenon was referred to as extra scour. However, the extra scour for revetments is assumed to be negligible, therefore only the structure induced scour, calculated by the method of Ahmad (1953), will be treated.

Ahmad (1953) developed a formula for the calculation of the equilibrium scour depth. It is described in Annex 7, Subsection 6.2 in more detail and reads for the revetment:

$$y_s + h_1 = K \left(h_1 \cdot u_1 \cdot \frac{B}{B-b} \right)^{\frac{2}{3}} \qquad (8.2\text{-}1)$$

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_1	=	upstream depth-averaged flow velocity	(m/s)
h_1	=	water depth upstream from scour hole	(m)
y_s	=	maximum equilibrium scour depth	(m)

In the wide and braided Jamuna river, the term u·B/(B-b) should be interpreted as the increased flow velocity near the structure. The predicted upstream depth average flow velocity u_1 reached more than 2 u_{crit}, therefore the latter value with u_{crit} = 1.0 m/s is taken for the application of Ahmad's formula (for details see Annex 7).

The measured flow depths and the depths of the scour hole are presented in Fig. 7.2-1. In the initial stage in June and July the scour hole developed slowly and the observed scour depth was not the equilibrium depth (see Subsection 8.2.3). At the time the maximum scour depth of about y_s = 14 m was reached (end of August and September 1997), the value of K, which describes the structure properties (alignment, porosity, etc.), was 2.1 to 2.4. The higher value is a safe first estimate for the design of this type of structures. In the final stage at the end of September and in October, the flow velocity decreased and the scour hole experienced sedimentation.

It is recommended to use Ahmad's formula with the factor for flow constriction and with K = 2.4. The formula should be applied with great care for changes in approach conditions due to the response of the river to the structure.

8.2.3 Time Dependent Development of the Scour Hole

The predicted equilibrium scour depth will only be reached after a certain period of more or less constant hydraulic and morphological conditions. Slow scouring may have the effect that the boundary conditions change before the actual maximum depth is reached. In this context the assumption of equilibrium scour is rather conservative. In contrary, very fast scouring processes may increase the risk of slides (see Subsection 7.5).

The scour depth in front of the revetment structure developed during the monsoon of 1997 and consequently is a function of time. If the actual depth of the scour hole y(t) is smaller than the upstream water depth h_1 (i.e., $y < y_s$), then the following empirical relationship can be applied (Hoffmans and Verheij, 1997):

$$\frac{y(t)}{h_1} = \left\{ \frac{t}{t_1} \right\}^{n_t} \qquad (8.2\text{-}2)$$

in which

h_1	=	water depth upstream from scour hole	(m)
$y(t)$	=	time dependent depth of scour hole	(m)
t	=	time	
n_t	=	empirical coefficient	(-)
	=	0.7 to 0.8 for revetment structures with 3-dimensional scour hole	

The time t_1 is defined as the time in hours at which the scour depth is equal to the upstream water depth (i.e., total water depth of 2 h_1 above the scour hole) and can be derived by the following empirical formula (Hoffmans and Verheij, 1997):

$$t_1 = \frac{330 \cdot h_1 \cdot \Delta^{1.7}}{\left(\alpha_t \cdot u_1 - u_{cr}\right)^{4.3}} \qquad (8.2\text{-}3)$$

in which

u_1	=	upstream depth-averaged flow velocity	(m/s)
u_{cr}	=	critical depth averaged flow velocity for initiation of motion	(m/s)
	=	0.6 to 1.0 m/s	
α_t	=	coefficient	(-)
		from 1.2 at the begin of the monsoon to 2 at the end of the monsoon	
Δ	=	relative submerged density of river bed sediment	(-)
	=	1.65	

The method has been applied to the 1997 development of the scour hole in front of the revetment structure, taking into account that sediment transport entering the hole from upstream reduces the scour depth. Unfortunately, the application has not yielded conclusive results for a general calculation method. Further investigation on the basis of more extensive measurements is recommended.

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