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MINISTRY OF WATER RESOURCES
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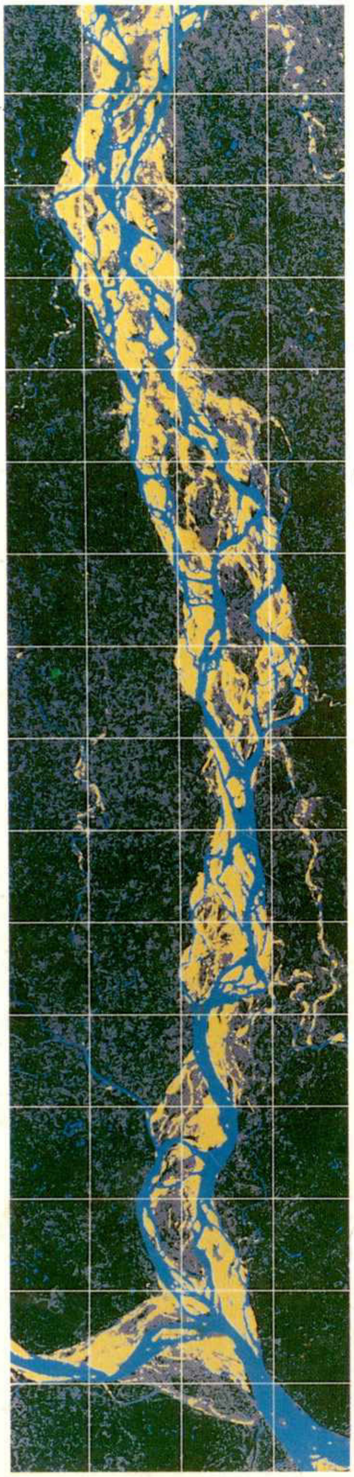

BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT FAP 21/22

EXTENDED STUDIES ON RECURRENT MEASURES (FAP 22)

KATLAMARI TEST SITE IMPACT AND BEHAVIOUR DURING MONSOON 1997

AUGUST 1998

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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2

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Table of Contents

Page

LIST OF TABLES

LIST OF FIGURES

LIST OF ANNEXES

1 INTRODUCTION

1.1 THE FAP 22 PROJECT

The River Training / Active Flood Plain Management (AFPM) Pilot Project is component 22 of the Flood Action Plan (FAP 22). Its final objective is to develop measures which would allow to control the Jamuna river in such a way that its outer channels would no longer threaten their outer banks by erosion. In addition, a reduced width of the river would increase the area of much needed cultivable land.

In the Final Planning Study Report of 1993 an alternative approach had been presented by the Consultant developed for replacing the classical approach to stabilize existing river banks or to narrow the river bed by applying "hard" measures for river training by a new approach considering that the river shall be "convinced, but not forced" to reduce its often threatening and unpredictable behaviour into a more gentle and predictable one. The application of "hard" measures contradicts the river's nature and includes the risk that as a result of the dynamic behaviour of the river the expensive permanent structures may lay idle for quite some time. Hence, as a result of the alternative study floating screens and artificial cut-offs were selected as most promising recurrent measures.

The study phase of FAP 22 was formally finalized in 1993 by holding an international experts discussion on the new concepts presented by the Consultant. The Experts unanimously agreed that the application of recurrent measures is promising since it makes use of the forces of the river which will maintain its dynamic character to some extent but in a more controlled way. Further studies and early testing of the proposed methods were strongly recommended by the Experts in order to be more specific about the technical and economic feasibility. Testing, however, should not be limited to laboratory tests only, but also prototype tests in the river were recommended in order to come to a firm assessment of costs and operation aspects. Moreover, the Experts expected the application of recurrent measures to be an economically attractive method to achieve a socio-economic development of those flood plain areas which are suffering from the lack of erosion control. It was furthermore recommended by them that the findings of the tests with recurrent measures should be included in the Master Plan for River Training in Bangladesh to be an integrated part of all river training works.

Finally, the Experts recommended not to limit further investigations to the recurrent measures suggested by the Consultant, but to go on with investigating into other methods also such as bandalling (fixed screens), intelligent dredging and channel cut-offs by low-cost earth works in parallel followed by further in-depth investigation on floating screens. It was stressed that a major aspect of all studies should be the examination of possible combinations of measures. Most of these methods were found to be attractive from a socio-economic point of view since they can make use of local materials and are labour intensive, hence making use of Bangladesh's cheap and abundant labour resources.

Based on the Experts' recommendations, the Consultant developed in 1994 a schedule of works for extended studies on recurrent measures followed by a proposal of technical and financial details for consultancy services and construction of recurrent measures in 1995. After the Consultant was requested by WARPO in March 1996 to take up recurrent measures activities, a proposal and workplan for the implementation of recurrent measures during the dry season 1996/97 and their testing during the monsoon season 1997 was presented. The actual activities started in September 1996 with selection of test sites. Early November it was decided to choose the bifurcation of the Fulchari channel and the Katlamari channel as the site for the 1996/1997 field tests of FAP 22 (Fig. 1.1 and 1.2). In the first months of 1997 the recurrent measures consisting of improved bandals and an earth dam were constructed at Katlamari Test Site and tested by the river during the monsoon 1997. In addition, in July 1997 some preliminary testing of two floating screens was performed. A monitoring programme was executed to measure the conditions of the river and the recurrent measures during the monsoon season 1997.

1.2 THIS REPORT

This report presents the evaluation of the recurrent measures as tested at Katlamari Test Site viz.
- improved bandals;
- an earth dam, and
- floating screens

The impact of these structures on the river and the behaviour of the structures during the monsoon 1997 and the following months are the subjects of this report. Some information on the morphological developments in the postmonsoon period has been included.

The impact and behaviour of the recurrent measures was monitored during the monsoon on a daily basis. The monitoring programme is outlined in the next chapter including the details of the executed measurements. The most important data are given in Chapter 4 and Annex B respectively.

The morphological impact of the structures is presented and discussed in Chapter 5 while Chapter 6 gives the analysis of the behaviour of the structures. The main conclusions from Chapter 5 and 6 are listed in Chapter 7.

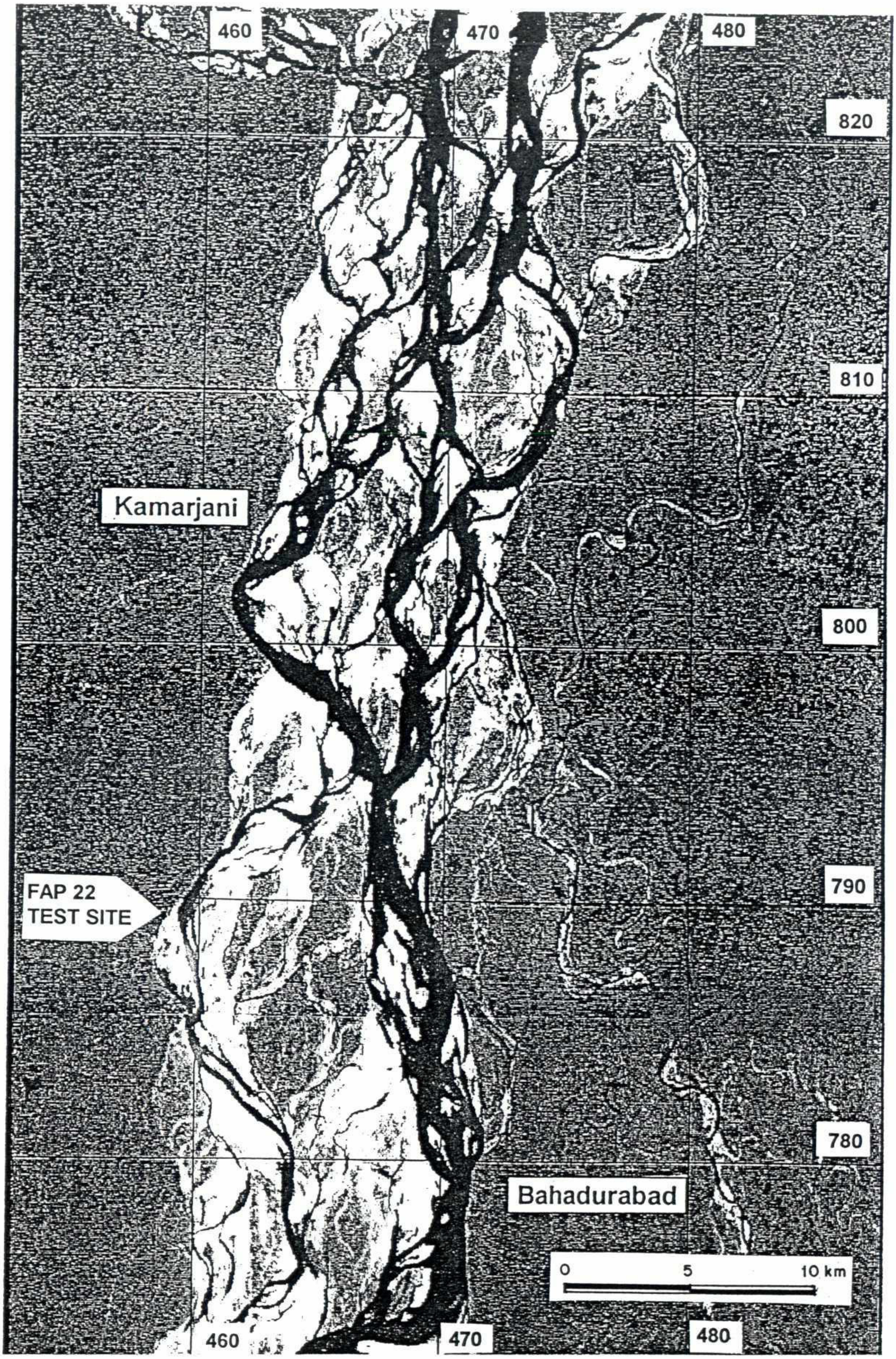

Fig. 1.1: Location of test site
(Landsat TM February 18, 1997)

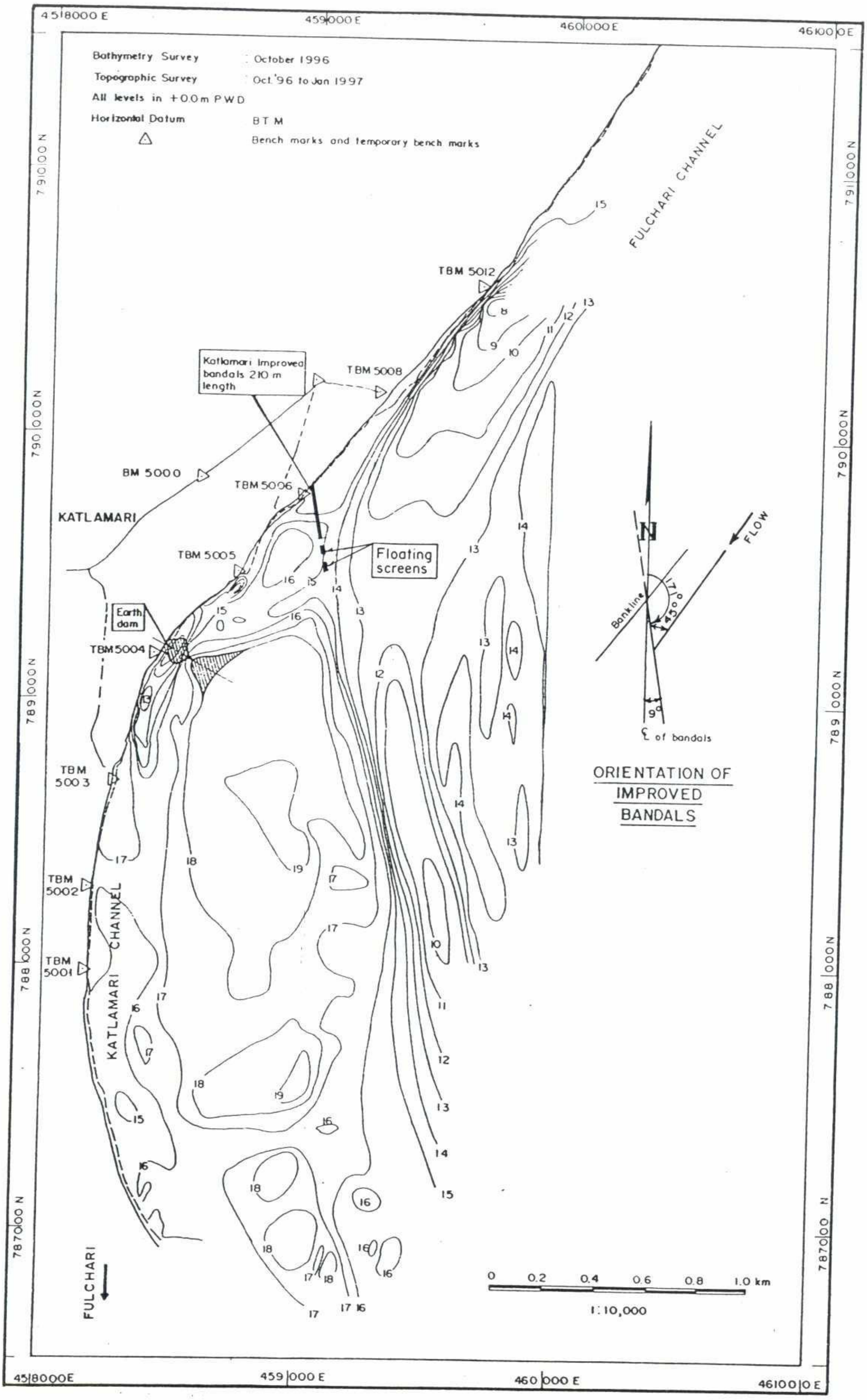

Fig. 1.2: Location of recurrent measures 1997

2 RECURRENT MEASURES

2.1 PRELIMINARY REMARK

The recurrent measures selected to be tested in the monsoon season 1997 consisted initially of an earth dam and improved bandals, whereas in the course of the season plans emerged to add some preliminary testing of floating screens. As in terms of timing the construction of improved bandals was rather critical, the work focused very much on the realisation of the bandals before the start of the monsoon rains in 1997.

The main activities of the 1996/1997 season are indicated in the following barchart. Some particularities of these activities are given in the following sections.

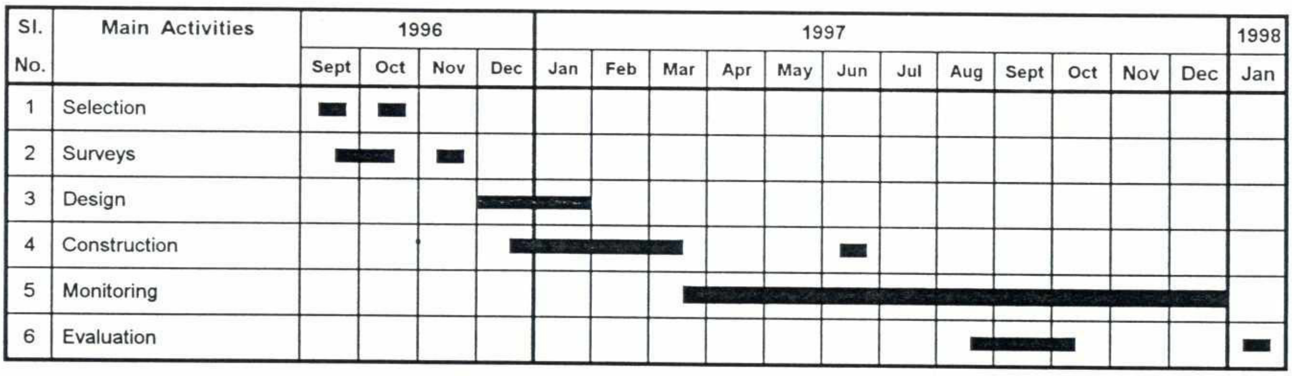

Fig. 2.1: Barchart of activities

2.2 SELECTION OF MEASURES

In the Experts discussion of 1993 [4] it was recommended to give priority to field tests especially regarding

- improved bandalling;
- intelligent dredging, and
- sills or plugs

Floating screens as recommended by the Consultant [1, 2, 3] were suggested to be developed in parallel or later.

After the final site selection in October 1996 it appeared that near Katlamari

- improved bandals;
- an earth dam, and
- floating screens

could be tested in the right anabranch of the Jamuna during the monsoon of 1997.

2.3 TEST SITE SELECTION

During each monsoon the river planform is changing considerably. During the postmonsoon these changes become gradually visible. Satellite images taken during the lowest water level reveal the full extent of these changes. Unfortunately, site selection for recurrent measures cannot wait for low water. Early selection is required to assess the location, type and extent of the measures in order to start timely with design and construction work.

72

The site selection started in the second half of September 1996 when the major part of the monsoon was over. The selection was realized in a number of steps:

- A first estimate of possible sites was made using 'old' premonsoon images of the river and information from interviews with WARPO and Railways staff. As a result 12 alternative test sites were identified;

- A reconnaissance survey was made from September 19 to 25 to the 12 alternatives test sites to assess the impact of the 1996 monsoon and to collect some data on channel dimensions;

- A multi-criteria evaluation was applied to preselected 3 sites out of the 12 alternatives. Criteria used were among other things impact, technical feasibility, cost, socio-economics and morphological predictability [8]. As a result the most promising sites appeared to be (see Fig. 2.2):
  1. Katlamari outflanking channel;
  2. Old Brahmaputra off-take, and
  3. Belgacha attached char

- From October 12 to 16 a survey was executed to collect more detailed information at the three sites and to verify the previous evaluation. A complicating factor was that the water levels in October 1996 were still high for the time of the year.

- The survey results and historical data (satellite images of the last 8 years) were used for further analysis. Especially the morphological (planform) developments were an essential topic in this study. The preliminary result was a clear preference for the Katlamari site.

- Results were presented to WARPO end of October 1996 and after a field visit of WARPO staff to the site it was decided to construct the FAP 22 recurrent measures in Katlamari.

Details of site selection are given in a separate report of August 1997 [39].

2.4 STRUCTURES

2.4.1 Preliminary Remark

Subsequent to the decision in favour of Katlamari as suitable test site, it was decided to build

- improved bandals at the off-take of the Katlamari channel with the aim to deflect the flow and to encourage sedimentation behind the structure, and
- an earthdam 600 m downstream of the bandal structure with the intention to close the Katlamari channel at the beginning of the flood season.

As to the design of the structures reference is made to the Construction Report of August 1997 [39].

2.4.2 Improved Bandals

The total length of the structure was 210 m with an orientation of 45° towards the expected flow of the river (see Fig. 1.2) The whole structure consisted of 4 main components of 4 m height above the char level at 15.8 m+PWD and 45 m length each followed by two components of 15 m length each and heights of 3 and 2 m respectively (see Fig. 2.3). The clearance between the char and the screens of the main bandals was two meters in all cases, whereas the top of the screens was at the expected mean flood level, thus closing the upper 50 % of the cross-section. The screens of the 4 main components were built over a length of 24 m of single wickerwork and of double wickerwork over a length of 21 m each. This allowed to test the influence of the permeability of the screens on hydraulic loads, wave loads and scour depths.

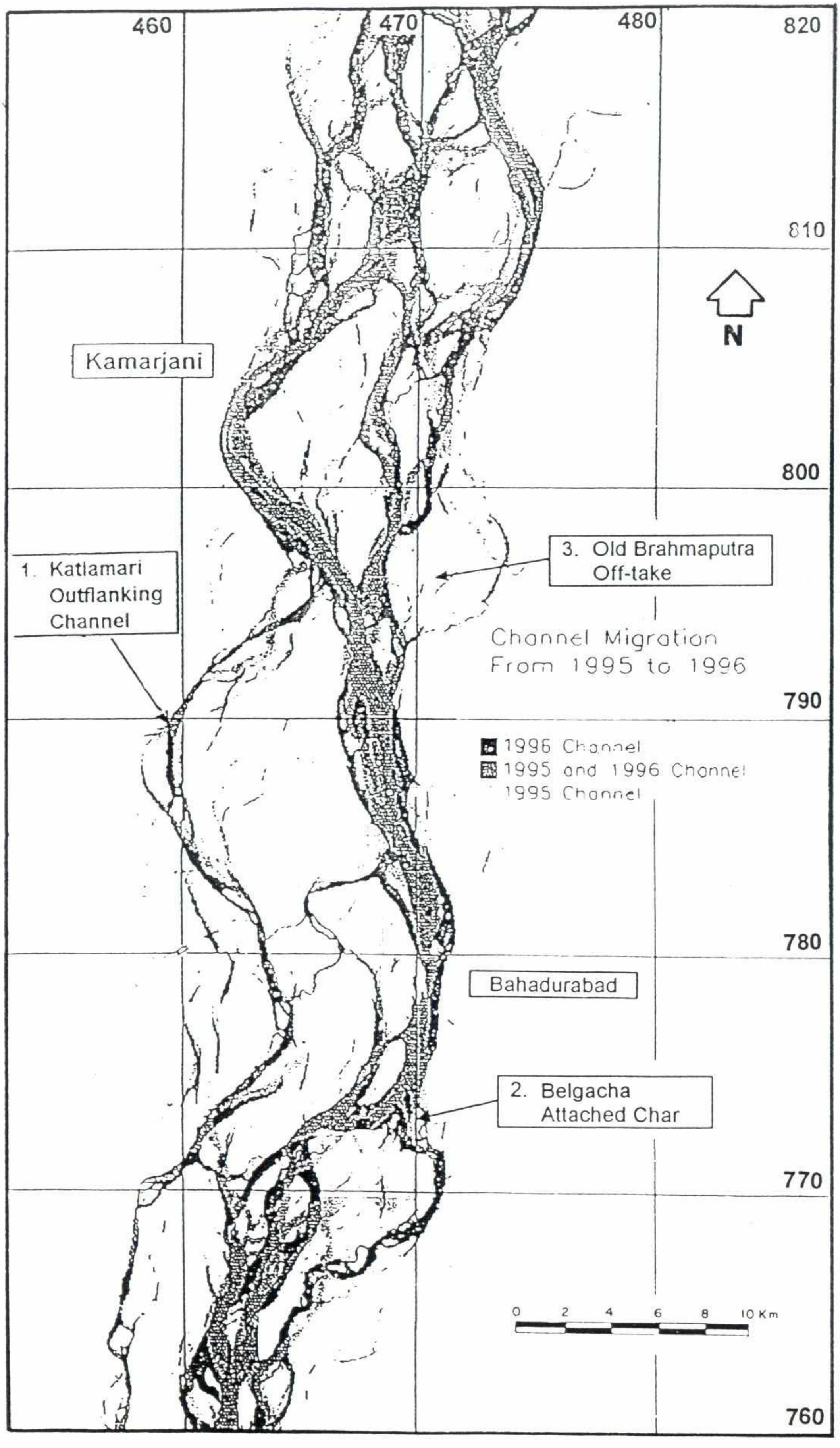

Fig. 2.2: Preselected sites

78

The foundation of the superstructure consisted of piles using bullah (tree trunks), bamboo and bamboo bundles with a spacing of the pile rows of 1.5 m in longitudinal direction.

The load assumption o the structure was based on measurements in the river branch and flow velocities computed in mathematical river model. The resultant loads were:

1.22 kN/m² hydraulic load, and
7.63 kN/m² wave load.

The scour depth below the screens was expected to be in the range of 3 to 4 m, which had to be taken into account for the design of the pile foundation.

Type A of the structure was designed for the hydraulic load and about 80 % of the wave load . The latter was reduced because its occurrence is rather rare on the one hand and the permeability and flexibility of the superstructure has to be taken into account on the other hand. Structural calculations resulted in a roof type shape of the superstructure and wooden piles of 7.5 m length for the foundation.

The structure **type B** was designed for the full hydraulic load and about 40 % of the maximum wave load only on a vertical screen of 2 m height. This component had a considerable flexibility, because the screen was only fixed at the upper part of the horizontal supporting bamboo.

The design of the structure **type C** was based on the consideration to maintain the flexibility of the superstructure, to use a minimum of costly materials like wood and a maximum of reusable materials such as ropes. The main advantage of this type was to have a minimum of foundation piles in the area of the expected scour hole. The structure was designed to resist the hydraulic load. In case of wave occurrence, the screens will start swinging thus minimizing the wave load. The stress in the prestressed ropes will change only little due to their high elasticity.

It was expected that the swinging screen of the structure **type D** would minimize the wave load considerably and thus the use of the supporting structure. The pile pattern of the foundation was the same as for the structure type B. However, the wooden tension piles were replaced by bamboo bundles resulting in only 50 % of the number of wooden piles. It was expected that the scour depth below the structure would be a minimum because the constriction of the flow underneath the screen will be the most favourable one of all types of structures as the submerged flow can smoothly follow the screen.

The elevation of the structure **type E** was reduced in order to minimize the development of eddies and thus scouring behind the bandals. There were two sections of 15 m length each. The 3 m high structure **type E 1** was founded on vertical bullahs only and the superstructure was a roof type like in section A. Since it was expected that the structure would be most of the time under water, the wave load was reduced again. **Type E 2** consisted of bamboo only fixed by wire and was, hence, the most simple one. Since the whole component was expected to be most of the time during the monsoon season under water, there was a good change to investigate the lifetime / decay of bamboo and the durability of connections.

2.4.3 <u>Earth Dam</u>

The location was selected to minimize the quantities and to maximize the effectiveness of the dam. As consequence the narrowest section of the Katlamari channel was chosen, which was about 600 m downstream from the bandal structure. The crest of the dam corresponded with the bottom level of the screens, so that the flow in the Katlamari channel was not totally blocked thus diminishing the effectiveness of the screens. The lower excavated char area (level at 17.4 m+PWD) was expected to attract slightly more flow, so that the velocities along the bankline were low thus encouraging the siltation of the channel. To prevent downstream cross currents and erosions a guiding dam separated the excavated char area and the dam. The crest level of this dam was 18.5 m+PWD, thus a little bit higher than the crest level of the earth dam.

The upstream and downstream slopes of the earth dam were 1:3 and 1:4 respectively. The crest width was 2 m. This was considered sufficient to withstand seepage pressure. The slopes of the dam and the adjacent river bank were protected by Hazen geojute and gunny bags. The geojute was placed under water using fascins and sinking by gunny bags. Above the water level the sheets were placed directly on the dam surface and then covered by gunny bags. The geojute sheets were sewn together, because they were rather small and overlapping would be too expensive. The excavated char area was protected from erosion by a sill made of two layers of gunny bags dug in along the centreline. This sill was connected to the protection for the dam and the guiding dam.

For details see Fig. 2.4.

2.4.4 Floating Screens

In order to gain some experience with floating screens two country boats were tied together in July 1997. Between the boats 5 screens of bamboo were installed which could be adjusted vertically and the dimensions of which were 3 m x 3 m (see Fig. 2.5). Two units were built and anchored as an extension of the bandal structure.

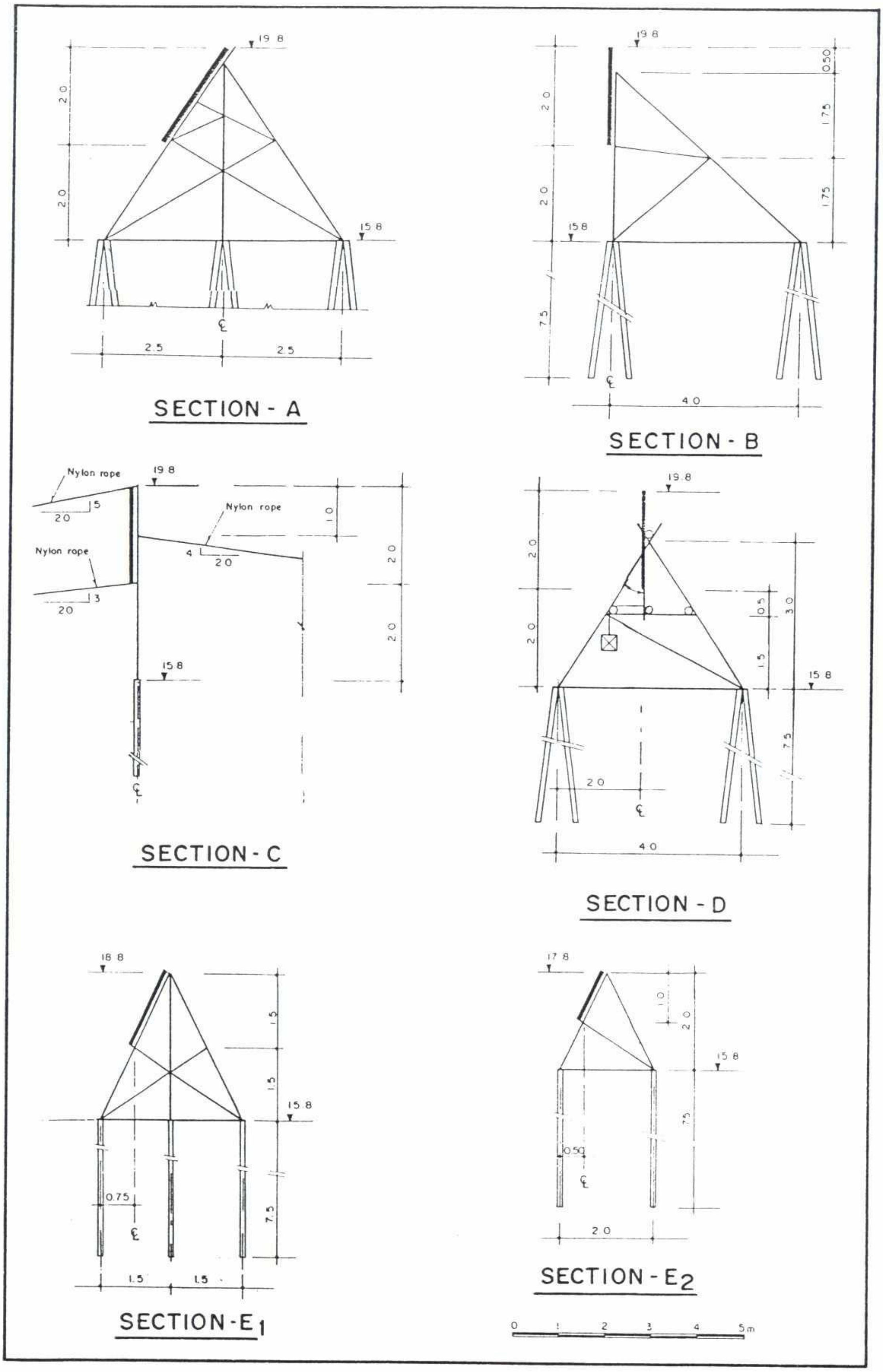

Fig. 2.3 : Design of bandal sections

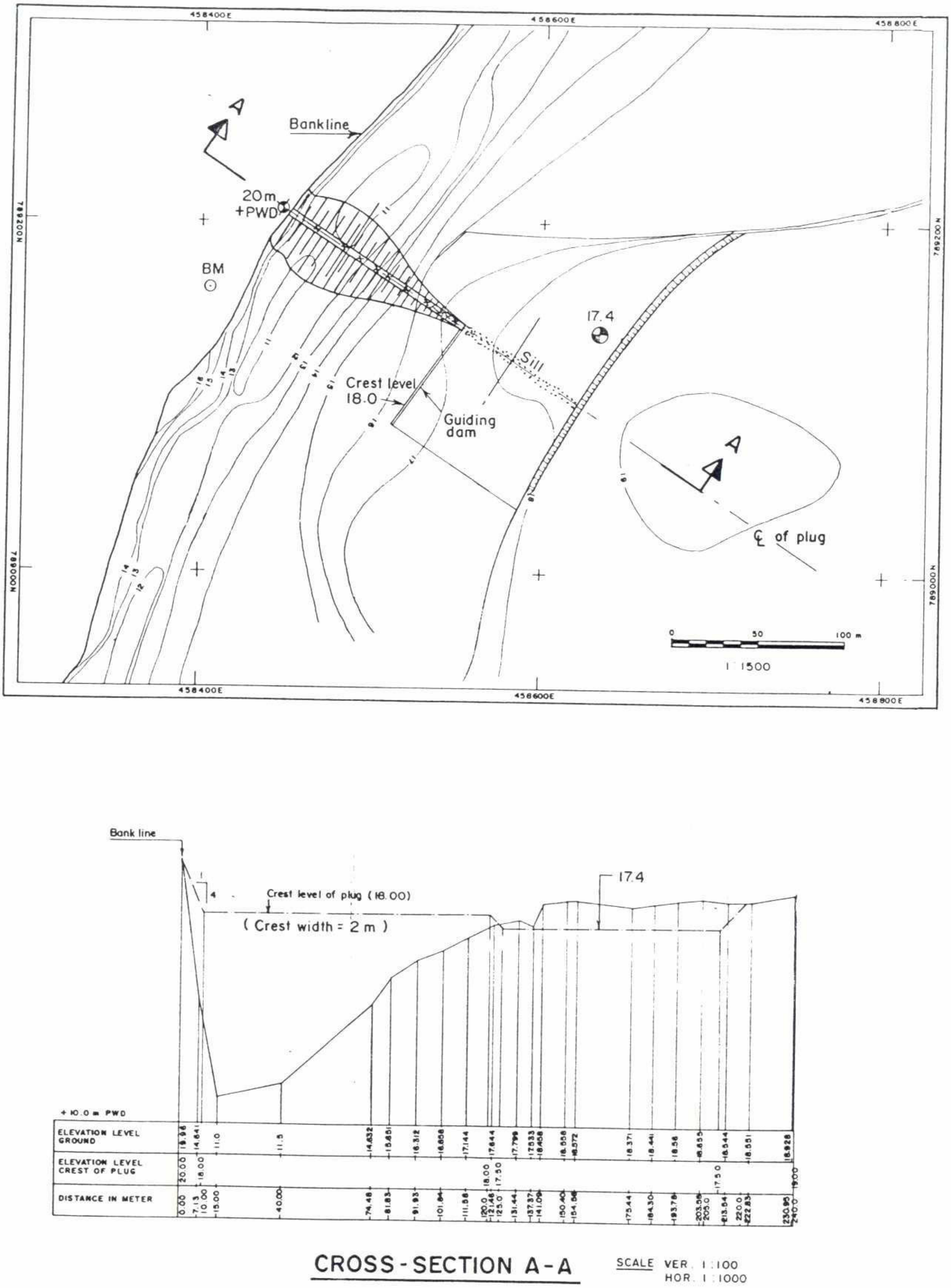

Fig. 2.4: Details of the earth dam

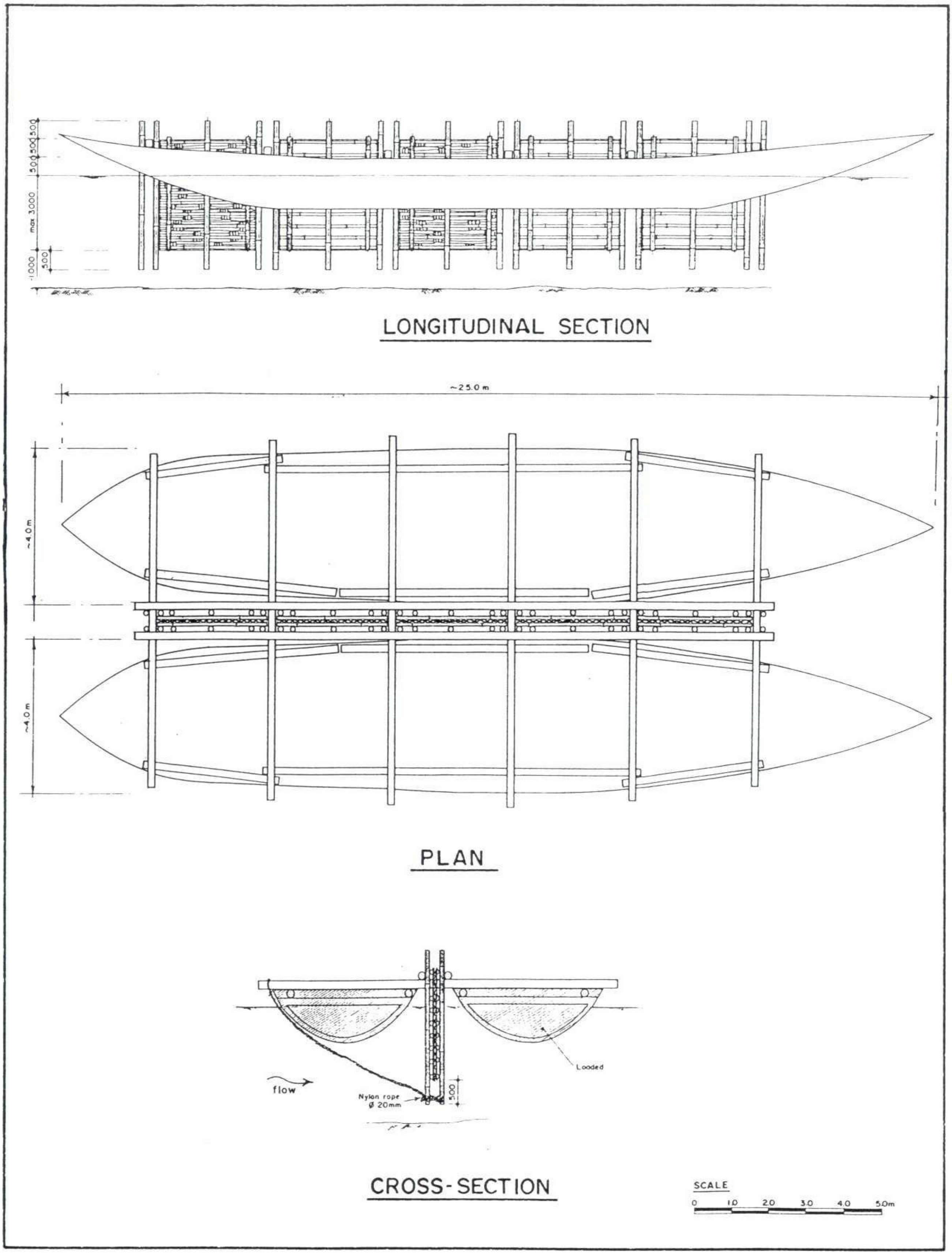

Fig. 2.5: Floating screens

3 MEASUREMENTS

3.1 SET-UP PROGRAMME

In October 1996 a monitoring programme was developed for the test structures at Katlamari Test Site.

The objectives of the monitoring were:

i) detect morphological changes, especially in the Fulchari channel (entrance, planform, deep channel sections), which may affect the selected test site near Katlamari;
ii) measure the hydromorphological conditions near the structures (water levels, velocities, waves, char levels, scouring depth and sediment transport);
iii) check the effectivity of the measures by measuring erosion/sedimentation especially in the Katlamari channel and check also the bank erosion in this channel, and
iv) check the river training works done by others in the Fulchari channel.

Based on these objectives, the physical changes in the river (for instance the speed of morphological changes) and the available survey capacity, the following programme was defined.

Parameter	Locations	Frequency
water level	3 stations	3 times per day
near field bathymetry	$\Delta x = 50$ m	2 per months
far field bathymetry	$\Delta x = 200$ m	1 per 2 months
bankline	right bank Katlamari channel	2 per month
bandal scour	under bandal	1 per day
flow velocity and direction	around bifurcation and near structure	1 per week 100s per point
wave height and direction	near bandals	during storm
sediment concentrations	together with some of the flow velocity verticals; 3 points per vertical	1 per week 300s per point

Table 3.1: Outline of the monitoring programme

The programme was specified in more detail especially regarding the positions of measurements (bathymetry, float tracking, velocity and sediment verticals, points in these verticals). In the course of 1997 the programme was slightly amended: the number and positions of the sediment verticals were changed and it was decided to measure also the bankline on the char and the flow velocities over the dam. Moreover, wind measurements were added to the programme.

To collect all kinds of site observations such as floating debris, behaviour of the structures, damages, etc. in an organized way, it was decided to design a logbook for the purpose. Each day a logbook sheet had to be filled in (see Table 3.2). The daily sheets were compiled into monthly logbooks.

Date: / /1997	Emergency message reported To: at: hrs.

METEOROLOGICAL DATA : Location Site Bandal Structure				
Time	8.00 hrs.	13.00 hrs.	17.00 hrs.	Storm during night?
Wind Direction				From:
Wind Speed				To:

WATER LEVEL DATA - Gauge readings [m +PWD]								
RATANPUR			KATLAMARI			FULCHARI		
8.00 hrs.	13.00 hrs.	17.00 hrs.	8.00 hrs.	13.00 hrs.	17.00 hrs.	8.00 hrs.	13.00 hrs.	17.00 hrs.

WAVE HEIGHTS (maximum readings from water level scale in cm)									
RATANPUR			KATLAMARI			NAVIGATION AID			FERRY
8.00 hrs.	13.00 hrs.	17.00 hrs.	8.00 hrs.	13.00 hrs.	17.00 hrs.	8.00 hrs.	13.00 hrs.	17.00 hrs.	

WAVE DIRECTIONS:									

VISUAL CONTROL OF STRUCTURES:						
Section	Damage Report Floating Debris Clean	Time		Wave Runup Rundown		Scour Depth [m +PWD]
		From	To			
A						
B						
C						
D						
E1						
E2						

SWINGING OF BANDALS SECTION D (heavy / strong / little / none)		
8.00 hrs.	13.00 hrs.	17.00 hrs.

INCLINATION OF BANDALS SECTION C:
Centimeters from Vertical (13:00 hrs.)

MONITORING SURVEYS CARRIED OUT:		
Bathymetry	Float Trackings	EDM Surveys
Line ID KAT 500 from: to: KAT 200 from: to:	Fulchari Ch. Katlamari Ch.	
Valeport Locations	Sediment Locations	Crest Velocity
		Location A: [m/s] Location B: [m/s]
Reported by: Asst. Hydrographer on Site	Checked by: Monitoring Team Leader	Seen: CSE or PM

Table 3.2: Daily logbook sheet

3.2 EQUIPMENT AND INSTRUMENTS

For the main part of the monitoring work FAP 21 survey boats have been used.

Specific issues were as follows:

- Water Levels
  Two staff gauges were installed near Katlamari and Ratanpur. Also nearby benchmarks were constructed. The original BWDB gauge readings of Fulchari appeared to be inaccurate, so it was decided to install there also a FAP 22 staff gauge. Gauge readings started on July 8, 1997. Staff gauges were also used to measure the head over the earth dam.

- Bathymetry
  A combination of DGPS, digital echosounder and navigation computer has been used identical to the FAP 21 equipment (Consulting Consortium, September 1997). For near field measurements a reference station at Katlamari (temporarily shifted from Bahadurabad) was used. The far field measurements were done with GPS only.

- Bankline
  The banklines were measured with the EDM total station. From June to mid August the "pathfinder" of the "Hochschule Bremen" became available. The pathfinder is a portable GPS system in a backpack. The antenna was sometimes mounted on a motorbike. The GPS co-ordinates of bankline and/or waterline are off-line corrected to DGPS accuracy. For these corrections the reference station of Bahadurabad was shifted to a suitable point at Katlamari.

- Scour
  Daily measurements of scour depths were partly done from the gangways on top of the bandals (sections A and B). For the remaining measurements a country boat was used. Depths were measured with a marked bamboo rod.

- Flow Velocity
  For flow verticals (velocity and direction) a Valeport 308 MKIII current meter was used (in direct reading mode) suspended from a cable [34].

  Float tracking was done with small floats with a resistance cross made of 0.5 m x 0.5 m tin plates. The cross was hanging at 0.5 m below the water surface.

  In July the special floats of the Hochschule Bremen were used. These floats have inbuilt GPS receivers. Off-line DGPS corrections are applied [33]. Also these floats were used at Katlamari with a resistance cross of 0.5 m x 0.5 m at 0.5m below the surface.

  Detailed flow measurements were done in the scour hole and around the floating screens using a hand held Valeport "Braystroke" BFM001 with a tail. The instrument was suspended with a cable from a country boat. Only velocities were measured, no direction. The depth was measured with a hand held echo-sounder.

- Wind
  At Katlamari a hand held anemometer was used for wind velocity measurements. Wind direction was estimated from a flag. At the FAP 21 test site at Kamarjani an ultrasonic anemometer (WNT) has been installed which measures wind velocity and direction [35].

- Waves
  Wave heights were read from staff gauges. An additional gauge was installed for that purpose on the navigation jack upstream of the bandals. The wave direction was estimated visually.

- Sediment Concentration
  Sediment concentrations were determined in the sediment laboratory of the River Research Institute (RRI) in Faridpur from suspended sediment samples. For this sampling a special pump-bottle system was constructed comprising a fish-shaped sinker, an intake nozzle, a tube and a pump (Jabsco 12V)

  The system was tested to find the right dimensions of tube length and diameter and the range of nozzle diameters in order to obtain appropriate flow velocities both in the tube and in the intake.

3.3 EXECUTED MEASUREMENT

Detail of the measurements executed at Katlamari are given in Annex A.

4 HYDROMORPHOLOGICAL DATA

The results of the monitoring programme in terms of survey data have been summarized and visualized in Annex B the content of which is as follows:

Annex B.1	Water Levels
Annex B.2	Flow Measurements
Annex B.3	Sediment Measurements
Annex B.4	Discharge and Sediment Analysis
Annex B.5	Wind and Wave Observations

As to the flow measurements reference is also made to the report on "Current Measurements at Kamarjani, Katlamari and Bahadurabad" of January 1998, carried out by a team of Hochschule Bremen from June till August 1997 and to the monthly Monitoring Reports.

Q8

5 MORPHOLOGICAL DEVELOPMENTS MONSOON 1997

5.1 ENTRANCE OF THE FULCHARI CHANNEL

The age of the Fulchari channel is more than 25 years. During this period the importance of the channel changed as well as the location of the off-take [39]. The Landsat image of February 1997 shows the premonsoon position and shape of the off-take (Fig. 1.1). The development of the off-take during the receding flood of 1996 can be illustrated by two bathymetric surveys of October 1996 and December 1996 done by Bangladesh Inland Water Transport Authority (BIWTA), (see Figs. 5.1 and 5.2). For positioning, BIWTA used the Decca Chain system and they plotted the data in feet with respect to the lowest low water level (LLW) in the chart. The FAP 21/22 bathymetric surveys at Katlamari cover up to the off-take of the Fulchari channel. But the spacing of the survey lines from Ratanpur to the off-take is 500 m. Ratanpur is on the right bank of the Fulchari channel and about 3 km upstream of Katlamari. Changes of the off-take during the monsoon can be assessed by using these surveys.

At the end of October 1996 the off-take of the Fulchari channel consisted of two channels, hereafter referred to as left branch and right branch. Both the channels were shallow and the minimum bed level was more than 1 m above the LLW. The length of the shallow part in the left branch was more than twice the length of the shallow part in the right branch. Considering the length of the shallow area, the water level differences along the channels and bifurcation angles increasing of depth by retarded scour was likely in the right branch. Bangladesh Railway engaged BIWTA dredgers in the right branch at the beginning of November 1996 for maintaining the railway ferry route in the Fulchari channel (Fig. 5.2). For the same purpose they constructed bandals in the left branch and at different shallow areas in the Fulchari channel. In November and December 1996 two cutter suction dredgers dredged 100,000 m^3 sediment in the right branch, but the result was not encouraging. On the other hand more than 2 m erosion of the river bed has been occurred in the left branch. From Fig. 5.2 it appears that the increasing depth was probably not the effect of the bandal, rather it was an autonomous development. One possible explanation of the increasing depth of left branch instead of right branch is that the main flow in the Kamarjani-Bahadurabad channel was closer to the entrance of the left branch than to the entrance of the right branch.

FAP 21/22 surveyed the bank line of the Fulchari channel on March 18, 1997, during which the water level at Katlamari was 15.09 m+PWD. The comparison of 15 m level contour lines of different surveys can be used for assessing the development of the off-take during the monsoon of 1997. Fig. 5.3 shows the changes of the 15 m level contour lines from March 1997 to July and August 1997. The left branch gradually silted up. The process started from upstream and continued in downstream direction. Simultaneously a channel close to the alignment of the right branch was gradually growing up. This process was expected continue up to the end of the monsoon. The bathymetry and flow pattern of the main channel were only partly known, (float tracking in the Fulchari channel on June 16 and August 10 only) therefore the processes of development of the right branch is difficult to explain.

By the end of 1997 the full change of the planform of the entrance of the Fulchari channel became visible from satellite imageries. The right branch of the entrance had completely been silted up during the 1997 monsoon. The left branch was still open but not accessible by boat in December 1997 (see Fig. 5.4 to 5.6).

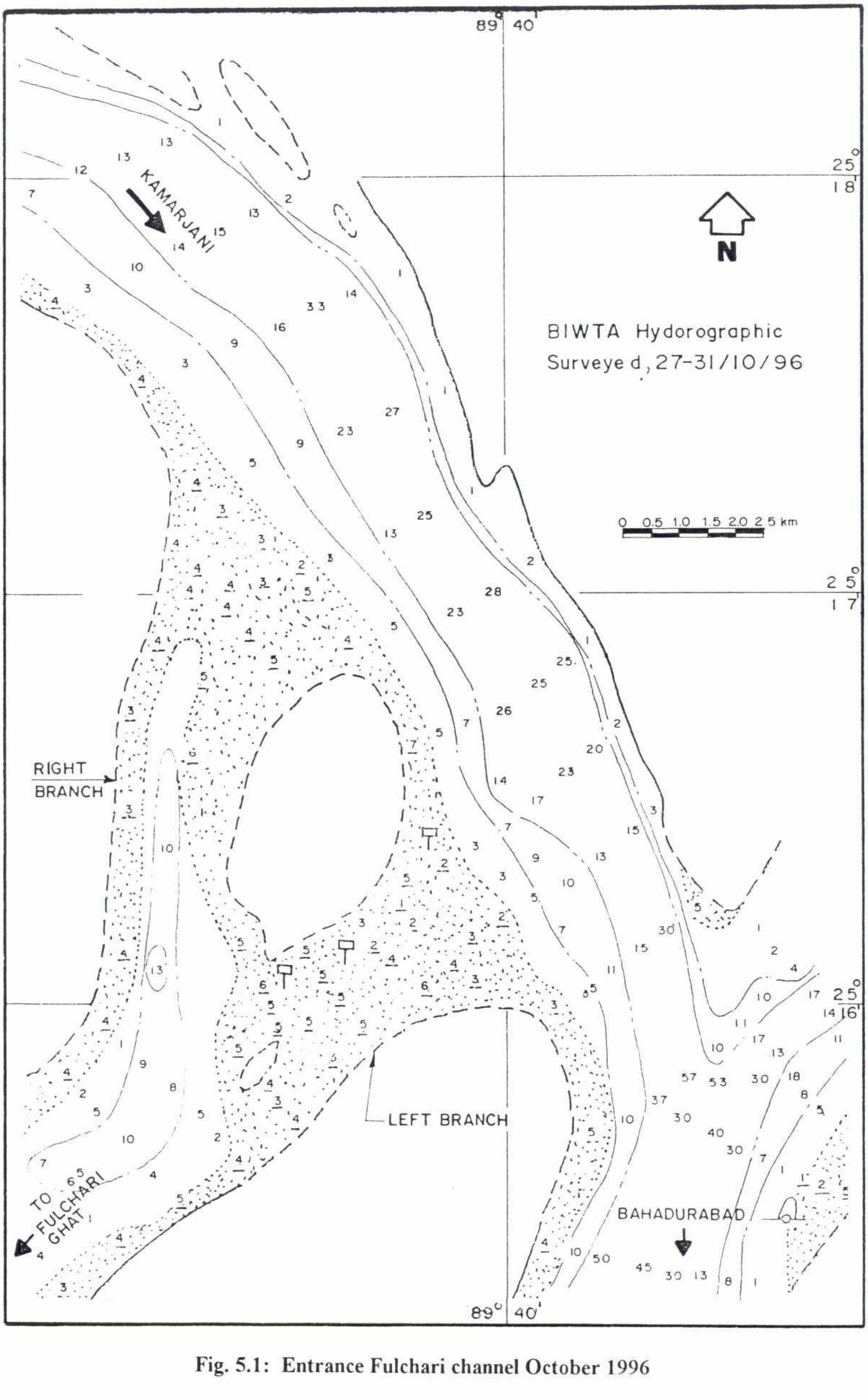

Fig. 5.1: Entrance Fulchari channel October 1996

26

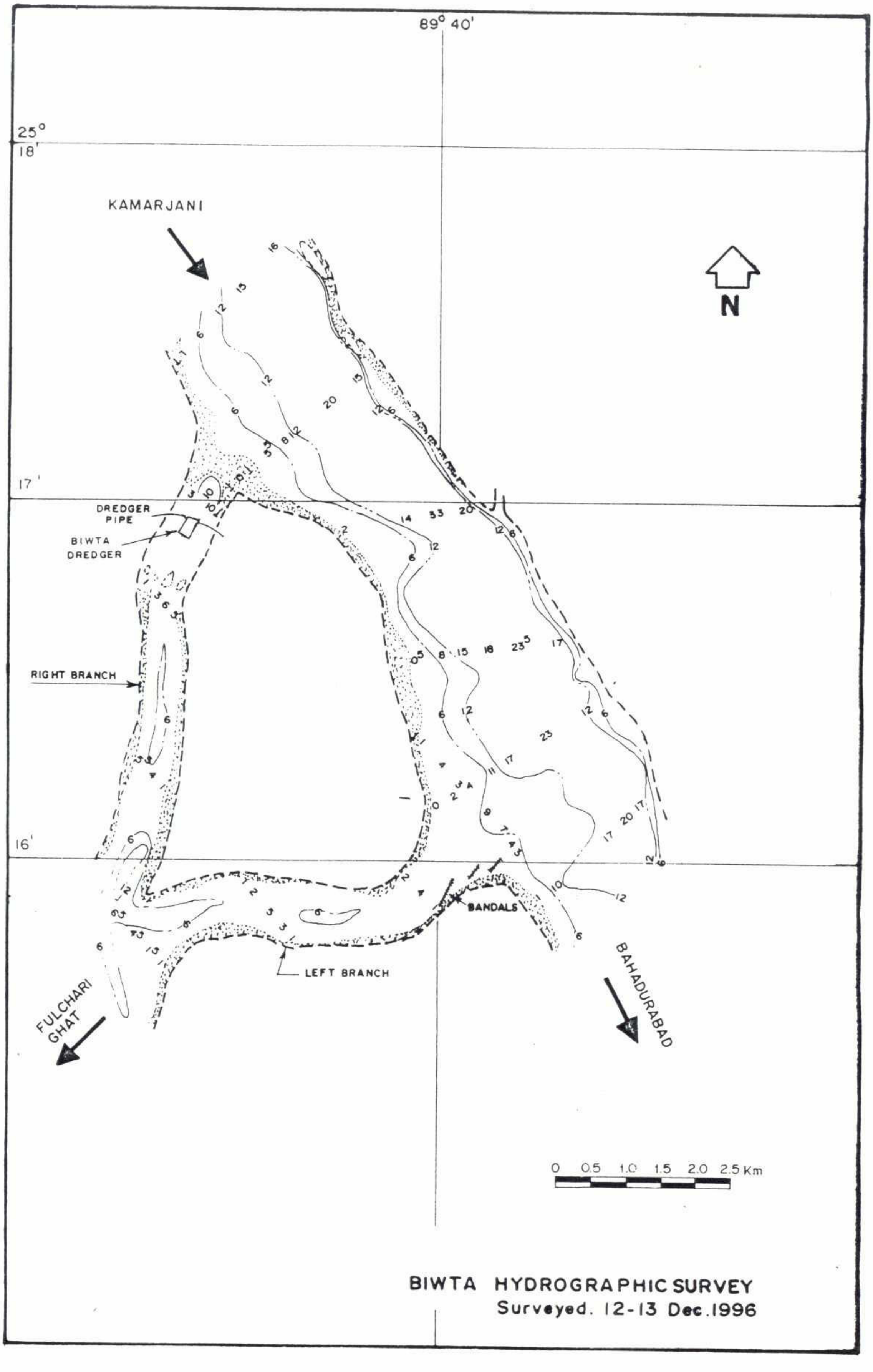

Fig. 5.2: Entrance Fulchari channel December 1996

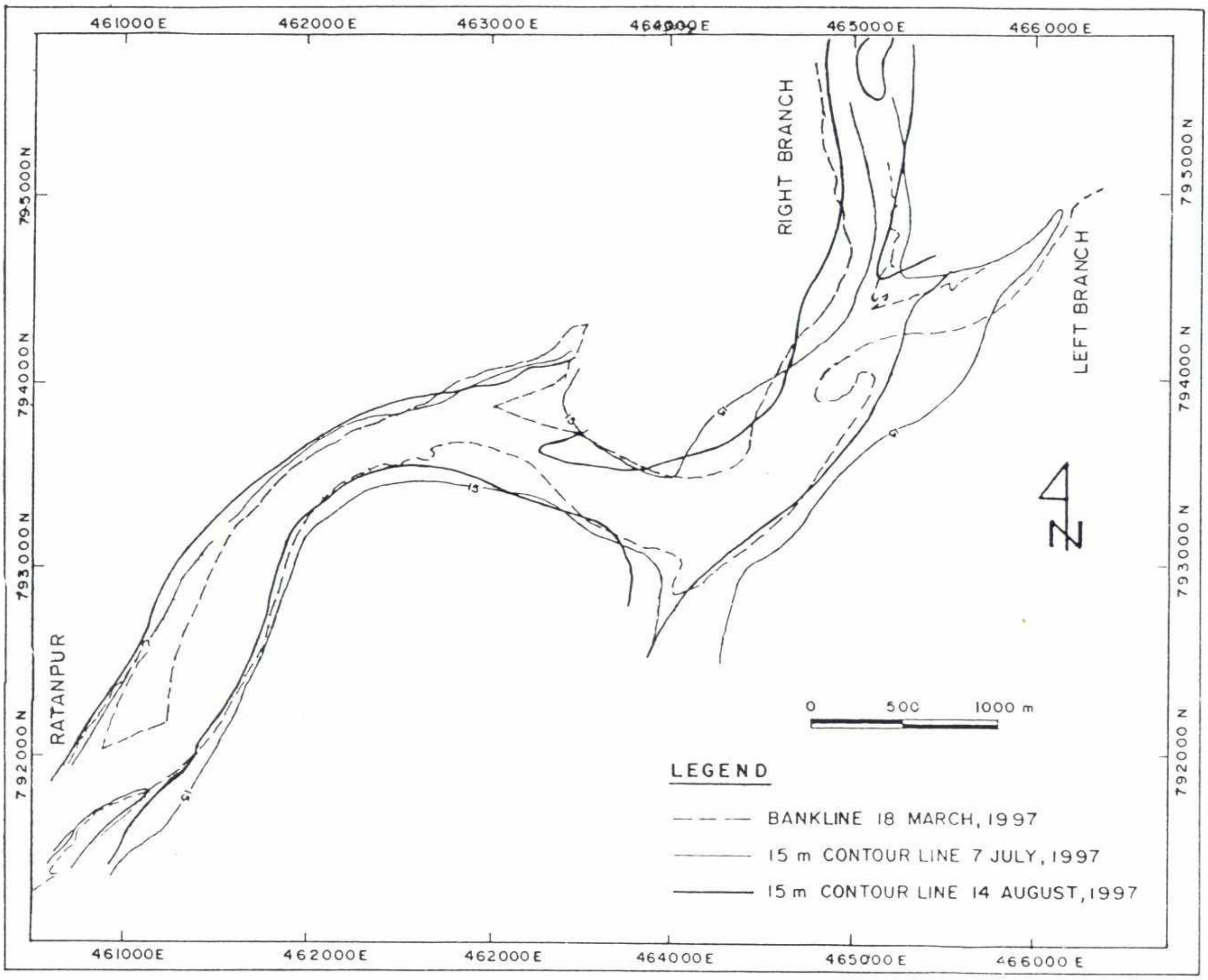

Fig. 5.3: Development Fulchari channel in 1997
(from FAP 21/22 survey data)

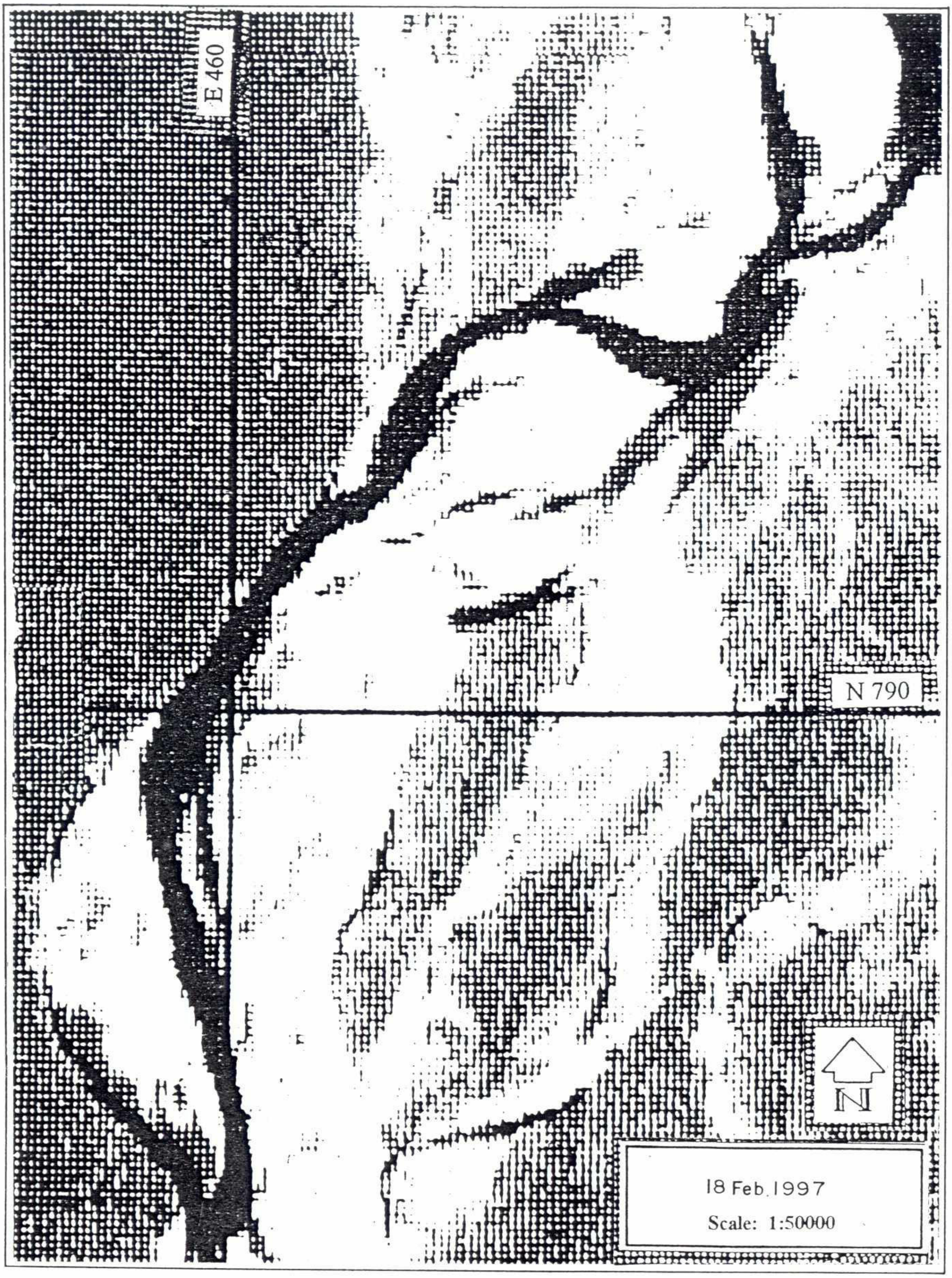

Fig. 5.4: Fulchari channel in February 1997
(from LANDSAT TM)

Fig. 5.5: Fulchari channel in December 1997
(from LANDSAT TM)

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Fig. 5.6: Difference map from February to December 1997 (from LANDSAT TM)

5.2 FULCHARI CHANNEL

It appears from the investigation of satellite images that the Fulchari channel had been declining gradually since the beginning of the nineties (Consulting Consortium, 1997). In the period 1993 to 1995 FAP 24 measured discharge and sediment transport in the Fulchari channel. FAP 24 measurements in 1995 are compared with FAP 21/22 discharge and suspended bedmaterial transport measured in 1997 (Fig. 5.7). The number of measurements in 1997 were not enough, especially during the high flow to conclude firmly about the magnitude of the decline of the flow and suspended sediment (sand fraction) transport in the Fulchari channel from 1995 to 1997.

The bathymetric surveys of FAP 21/22 can illustrate the changes with time from downstream from Ratanpur to near Fulchari Ghat. Fig. 5.3 provides the impression about the changes from the off-take to Ratanpur. The changes of the planform of the Fulchari channel during the monsoon is clearly visible from Fig. 5.6.

The channel thalweg in the river stretch from Ratanpur and Katlamari was parallel and near the high bank in November 1996 (Fig. 5.8). From bathymetric charts of that date it appeared that the deep scour hole level of 8 m+PWD was more than one kilometer upstream and the thalweg was about 400 m away from the bandal. Instead of parallel to the high bank, the channel thalweg was approaching in 1997 towards the bank at an incidence angle of about 10°, hits the bank about 500 m upstream from the bandal. As a result a deep scour hole of 6 m+PWD was very close to the bandal and the thalweg was only 200 m away from it. The thalweg gradually shifted west causing erosion at the upstream side of the Katlamari char. This erosion resulted in lateral accretion at the downstream edge of the char.

Date	Water level Katlamari [m+PWD]	Water level Bahadurabad [m+PWD]	Discharge [m^3/s]	Sediment transport [kg/s]		
				Coarse	Fine	Total
26/06/1997	18.63	17.89	5261	269	1980	2249
11/07/1997	20.01	18.99	7258	1571	5325	6896
28/07/1997	18.38	17.62	4683	594	1892	2486
14/08/1997	19.03	18.23	6104	294	4028	4322
05/09/1997	17.68	17.29	3708	343	1597	1940
21/09/1997	19.36	18.92	6139	1044	3376	4420

Table 5.1: Flow and sediment transport measurements in the Fulchari channel

5.3 KATLAMARI CHANNEL

During the dry season of 1996/1997 there was no flow in the Katlamari channel. The entrance of the channel was dry. Moreover, construction of the earth dam impeded the flow into the channel until the water level overtopped the adjacent char to the earth dam on June 12, 1997. At the entrance the flow was partly deflected by the bandals and was entering into the Katlamari channel under and downstream from the bandal structure. Gradually a char built up just downstream from the bandal and this char forced the flow to make a concave curve while entering into the channel, resulting in erosion at the upstream tip of the Katlamari char.

Date	Water level Katlamari [m+PWD]	Flow area [m²]	Discharge [m³/s]	Sediment transport [kg/s]		
				Coarse	Fine	Total
26/06/1997	18.63	674	554	41	250	291
10/07/1997	19.76	1260	1041	297	897	1194
28/07/1997	18.38	758	441	44	137	181
14/08/1997	19.03	1044	660	72	443	515
06/09/1997	17.85	598	323	36	120	156
22/09/1997	19.33	1146	830	105	452	557

Table 5.2: Flow and sediment transport measurements in the Katlamari channel

5.4 KATLAMARI VS FULCHARI CHANNEL

The flow and sediment transport measurements (Tables 5.1, 5.2 and Fig. 5.9) show that sharing of water between the Katlamari channel and the Fulchari channel increased as water level rose. It appears from Fig. 5.9 that the stage discharge relation of both the Katlamari channel and the Fulchari channel did not change substantially over the monsoon. The average velocity in the Katlamari channel was almost half of the average flow velocity in the Fulchari channel. But the sediment concentration in the Katlamari channel was higher than in the Fulchari channel. This is probably partly due to the erosion in the entrance of the Katlamari channel just upstream from the gauging section (Fig. 5.10) and partly due to higher sharing of sediment of a bifurcating channel with higher bifurcation angle. However, the flow entering with higher sediment concentration in the Katlamari channel caused deposition in the channel downstream and to the adjacent char (Fig. 5.11).

5.5 KATLAMARI CHAR

The bifurcation of the Katlamari channel shifted downstream resulting in erosion of the upstream part of the Katlamari char. Moreover, the gradual migration of the Fulchari channel eroded the upstream eastern part of the char. The changes of the char during the monsoon can be better assessed by comparing the November 1996 survey and a survey after the monsoon 1997 consisting of river bathymetry and land survey on the char. In absence of this type of survey comparison of char line surveys and bathymetric surveys at the beginning and end of the monsoon can serve the purposes partly. Fig. 5.11 shows that the char was expanding in lateral and downstream direction. The char was vertically accreted also (Fig. 5.8).

5.6 IMPACT OF THE MEASURES

The morphological impact of the improved bandals was considerable. Downstream from the bandals a char developed (Fig. 5.10). In order to quantity this process a certain area behind the bandals was selected and from the bathymetric data of all near field surveys the changes in terms of m^3 of erosion and sedimentation were computed. The results are presented in Fig. 5.12. For details of the morphological changes reference is made to Annex C where contour plots of the near field surveys are given with various types of difference maps.

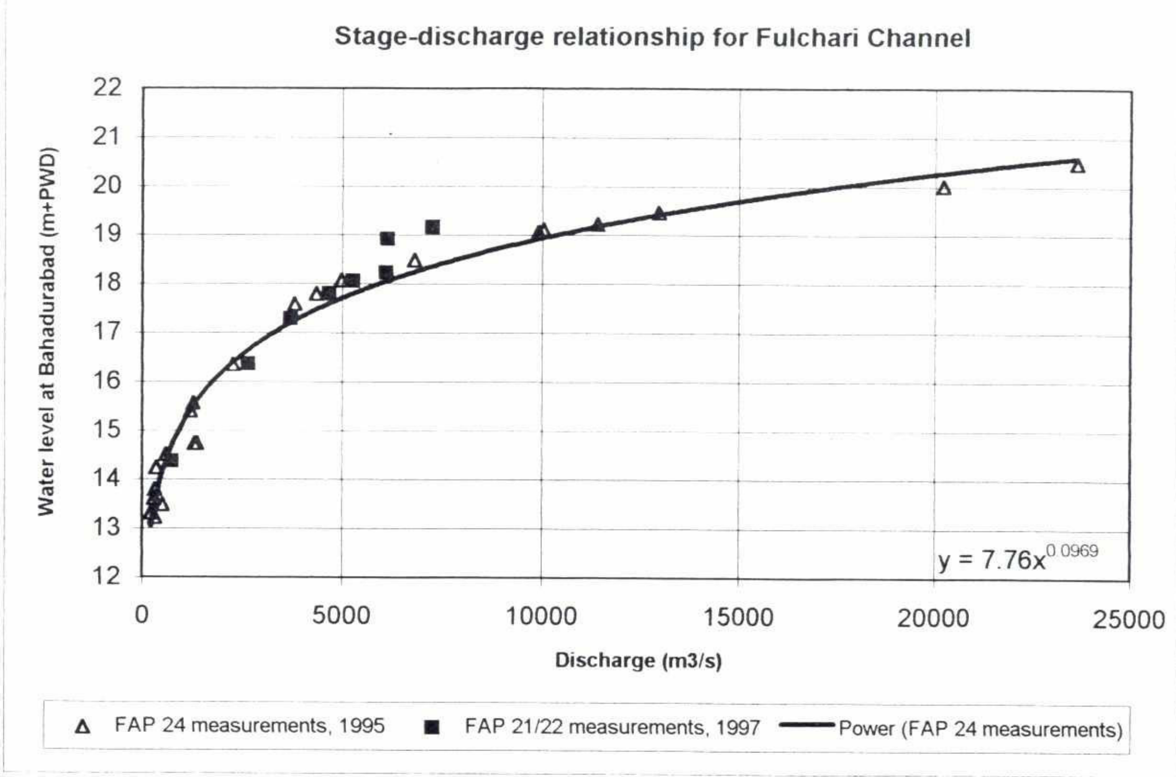
5.7 COMPARISON WITH PREDICTION MADE IN THE DRY SEASON 1996/97

In the dry season of 1996/1997 predictions for the developments of the Katlamari channel were made for two purposes: (1) for safety of the structural measures and (2) to assess the effectiveness. For the first case predictions were made on the basis of Consulting Consortium (1993 [40] and 1997 [38]) by using satellite images and FAP 21/22 survey data [39]. For the later case, predictions were made by mathematical computations using WENDY (11).

There are four main aspects in the predictions made for the monsoon 1997 on the basis of Consulting Consortium (1993 [40] and 1997 [38]):

(1) the probability that the Fulchari channel would not be abandoned was 100 %, although the channel was gradually declining;
(2) the probability that the main flow of the channel would be in the same channel as it was in 1996 was 100%;
(3) the probability that the Katlamari channel would be opened as it was in 1996 was 50% without any structural measures, and
(4) this probability was expected to decrease for the case with structural measures.

The development of the off-take and the Fulchari channel during the monsoon 1997 shows that the first two predictions were okay but it was difficult to say what could happen in the Katlamari channel without structural measures. The importance of the Katlamari channel in carrying discharge did certainly not increase during the 1997 monsoon. It seems that the channel was gradually sharing less water in favour of the Fulchari channel.



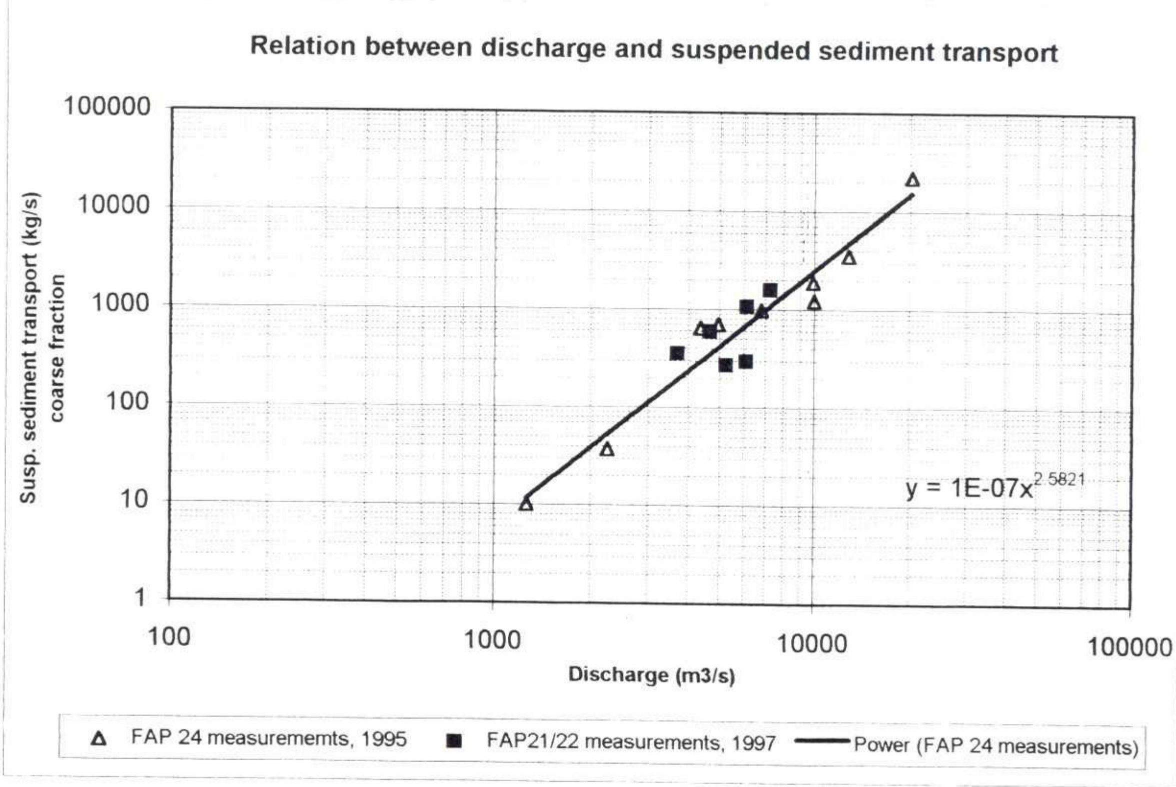


Fig. 5.7: Relation curves Fulchari channel

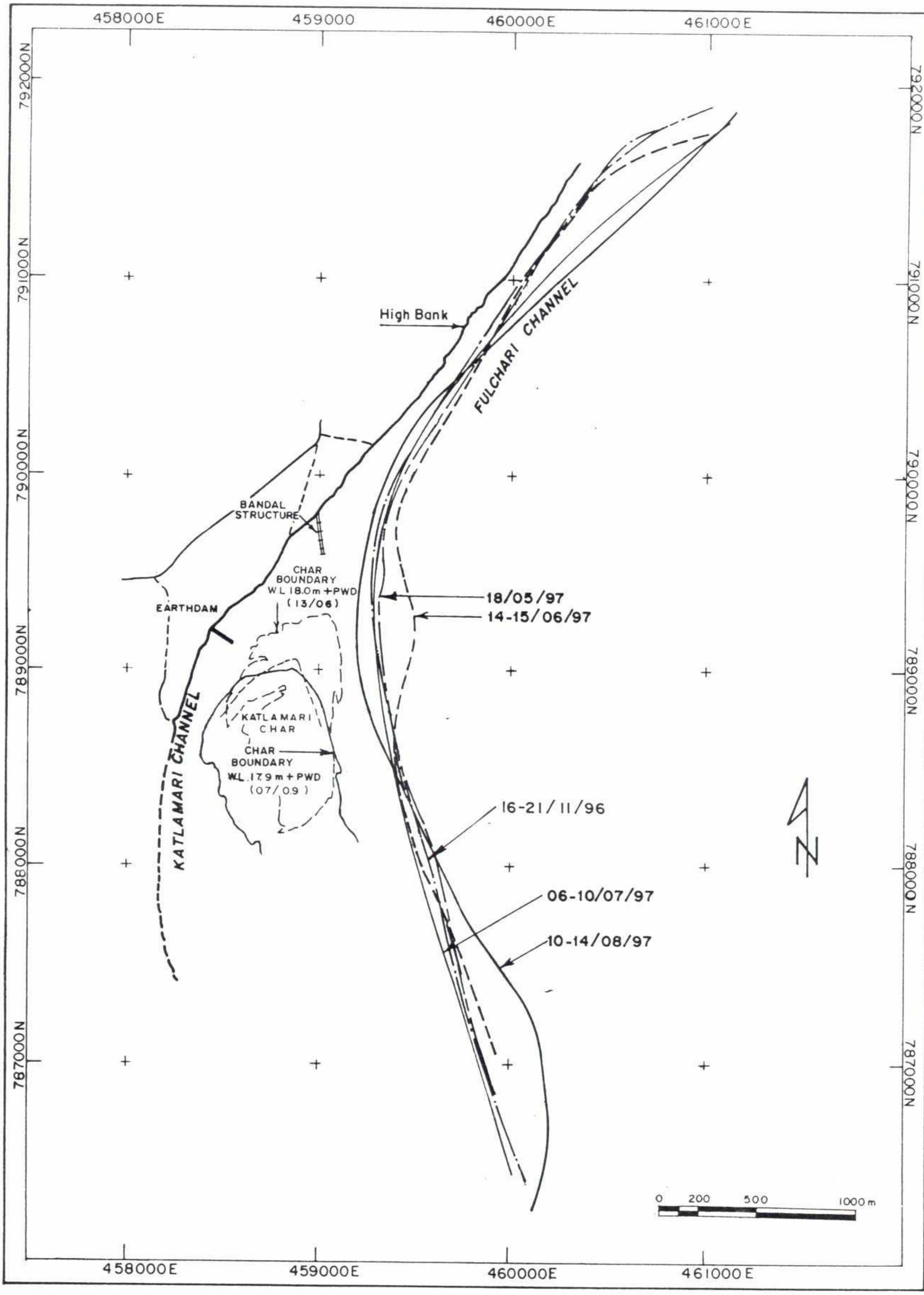
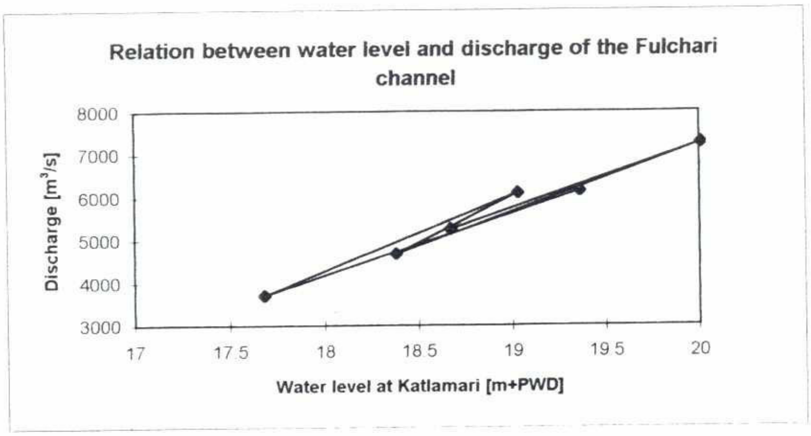
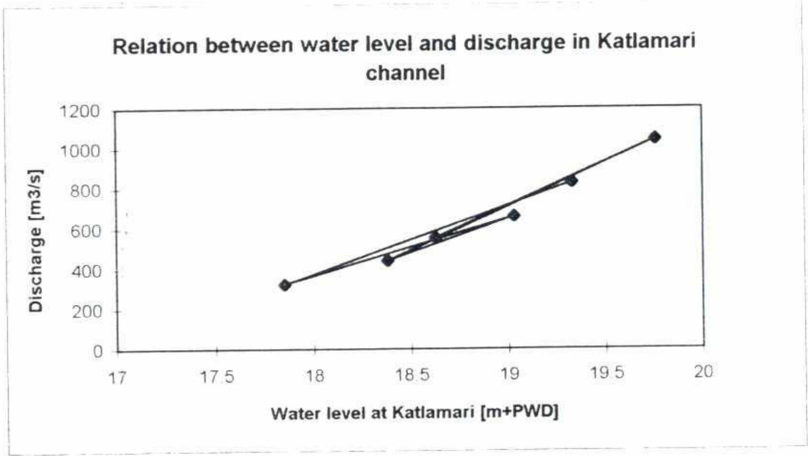
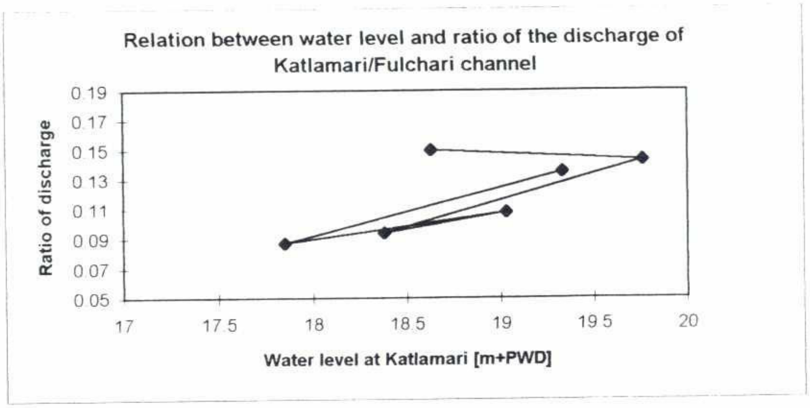


Fig. 5.8: Shift of thalweg of Fulchari channel in 1997 (estimated from FAP 21/22 bathymetric charts)







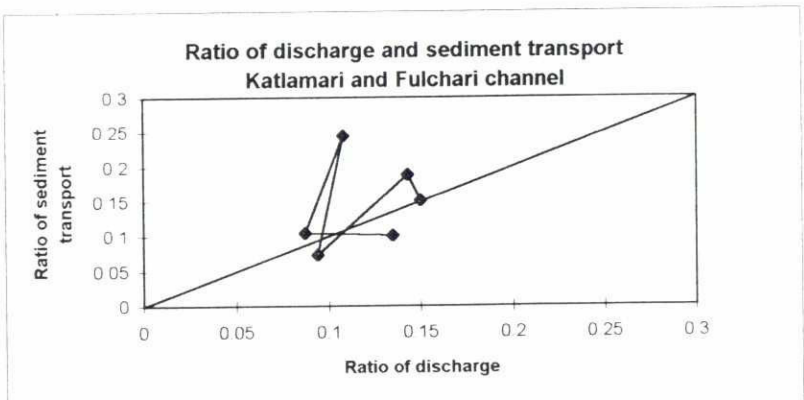


Fig. 5.9: Discharge and sediment transport relations between the Katlamari and Fulchari channels

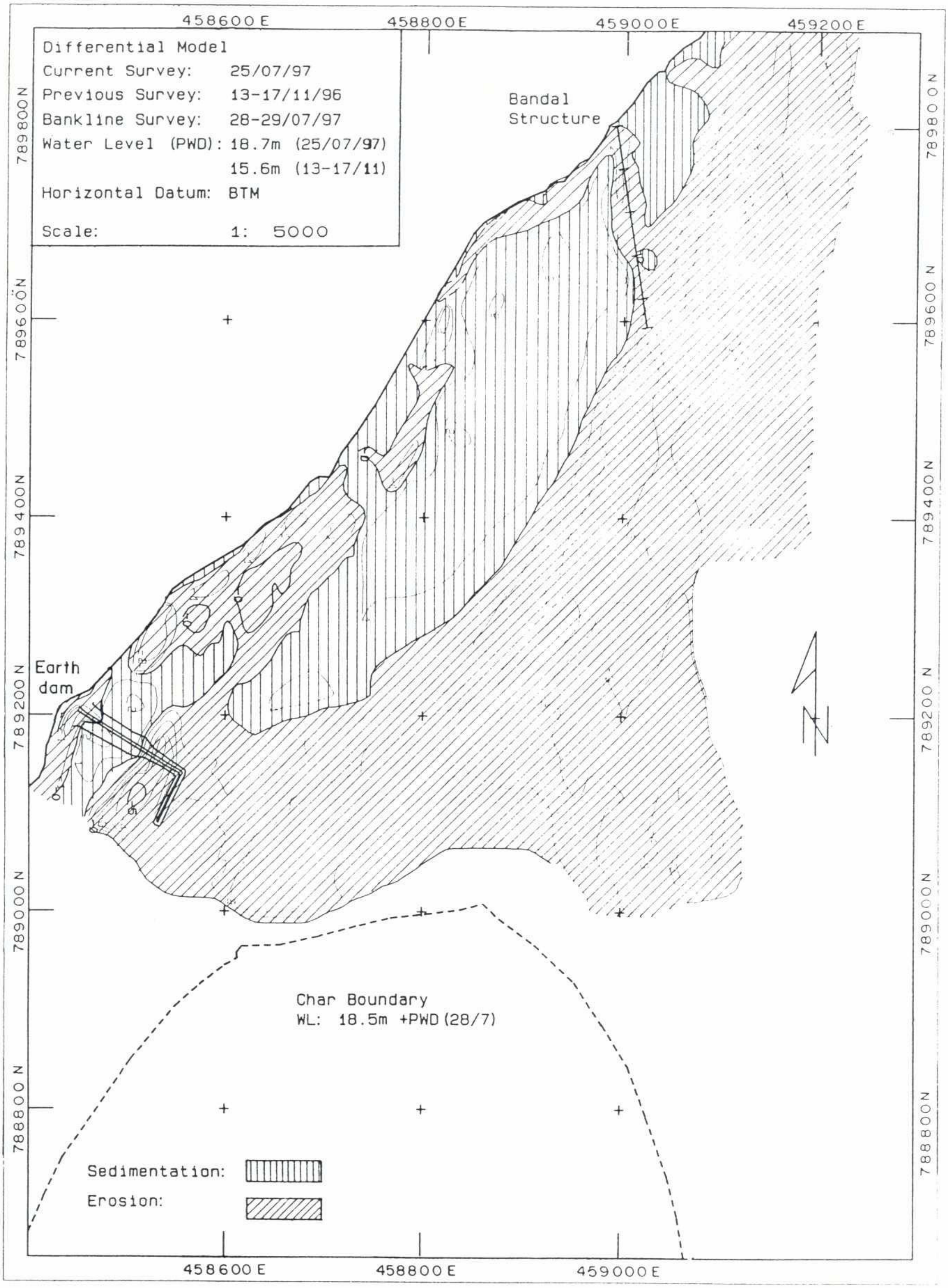


Fig. 5.10: Erosion and deposition at the off-take of the Katlamari channel in the period of November '96 to July '97

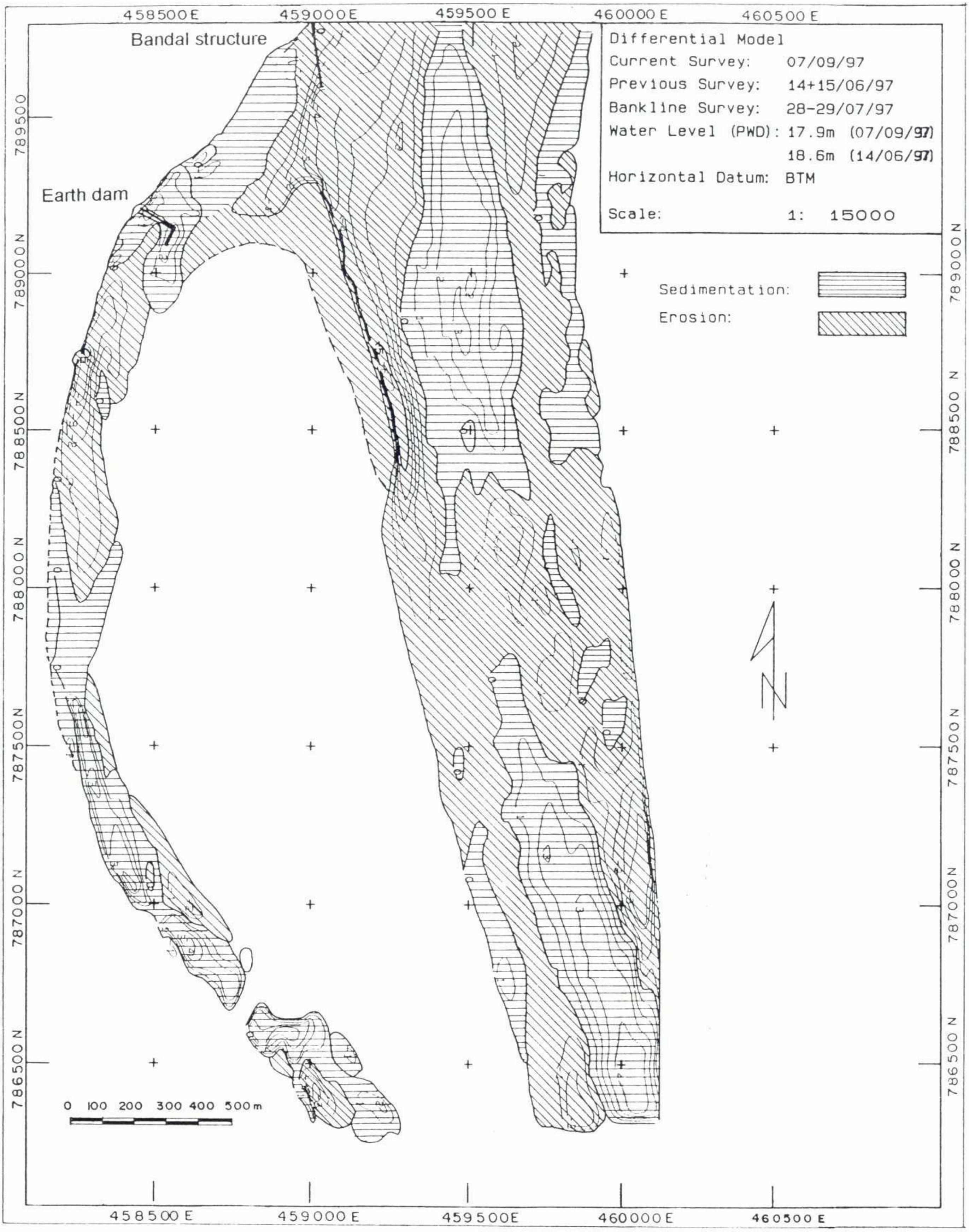
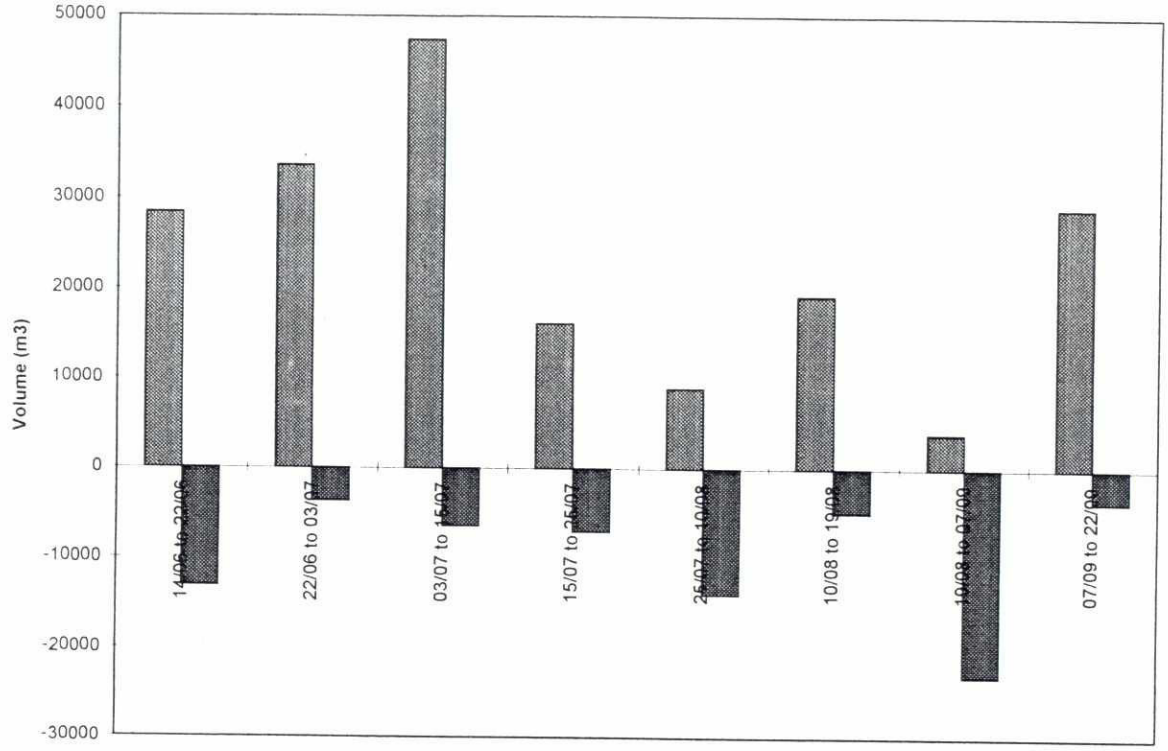


Fig. 5.11: Erosion and deposition around the Katlamari char in the period of June to September 1997

Sedimentation and Erosion



Cummulative Sedimentation at Katlamari Channel

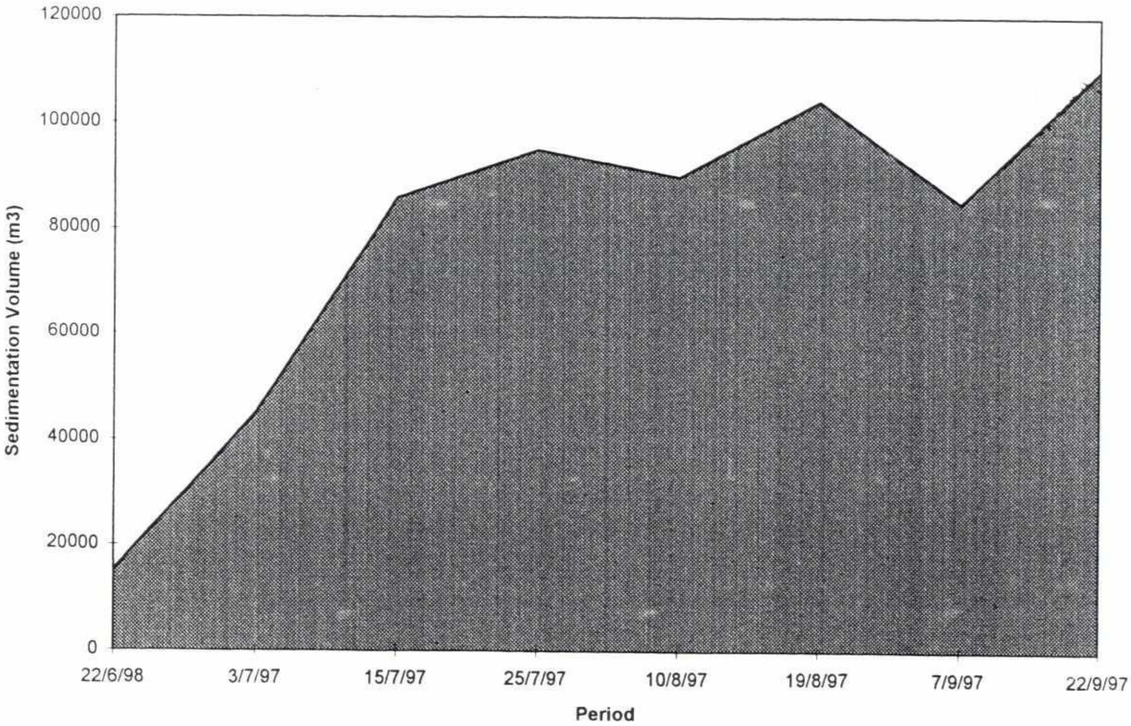


Fig. 5.12: Morphological impact of bandals

6 BEHAVIOUR OF THE STRUCTURES

6.1 INTRODUCTION

Obviously there is a lot of interaction between the recurrent measures and the river. As the purpose of the measures is to induce changes in river morphology by affecting the flow and sediment transport, the effect of the measures on the river seems dominant (see Chapter 5). However, the inverse effects, viz. the impact of the river on the measures is equal important. It is specially important in view of the design of the structures.

Therefore in this chapter the main subjects are: i) the general behaviour of the structures during the monsoon of 1997, ii) the local scour around the structure, threatening to destabilise the structure; and iii) comparison of the actual detailed hydraulic conditions near the structures with the design conditions estimated in 1996. These subjects have been discussed (as far as relevant) for the three type of measures applied near Katlamari:

- the improved bandals;
- the earth dam, and
- the floating screens.

6.2 BANDALS

6.2.1 General Behaviour

The bandals survived the monsoon of 1997 in good condition. The maximum scour depth as measured between Sections B and C appeared more than expected viz. 5.1m instead of 4.2m (3.8m bandal scour plus 0.4m pile scour), but did not destabilise the structure (see next subsection). Minor damages occurred mainly to the screen matting (see Subsection 6.2.3). The deeper parts of the Fulchari channel approached the bandal site gradually. In September 1997 the channel came close to the tip of the bandals and Section E2 was washed away. During the last peak of the monsoon in the last week of September 1997, further erosion near the bandal tip took place and also Section E1 was washed away. Also some differences were noticed in the behaviour of the different types of bandals (see Subsection 6.2.2).

The water levels were rather high in the monsoon of 1997, however, the wave height appeared to be moderate in the Fulchari channel. Sometimes the waves generated by the nearby passing railway ferry seemed more destructive than the wind waves. For comparison between design loads and measured conditions see Subsection 6.2.4.

The purpose of the bandals: to protect the downstream bank by promoting sedimentation was achieved. Sedimentation up to 3 m was measured. For the morphological impact see Chapter 5.

6.2.2 Local Scour

(a) Introduction
As soon as the rising water levels flooded the bandal site, scouring started around the pile heads and other parts of the bandal structures close to the char level of 15.8 m+PWD. However, as anticipated, this scour was very minor. The major part of the scour was expected to be generated by the screen and therefore scour measurements started in the rising limb before the main bandal screens were submerged for about 50% of their height (June 14, 1997, water level about 18.6 m+PWD). From that date daily measurements were taken till September 30, 1997. As the measurements were done from the top of the structure, during extreme high waters they were interrupted, especially at the sections near the Fulchari channel (C, D, E).

For details of time and location please refer Annex A.

80

(b) Scour Depth in Katlamari

Results of the depth measurements have been summarized in the following graphs:

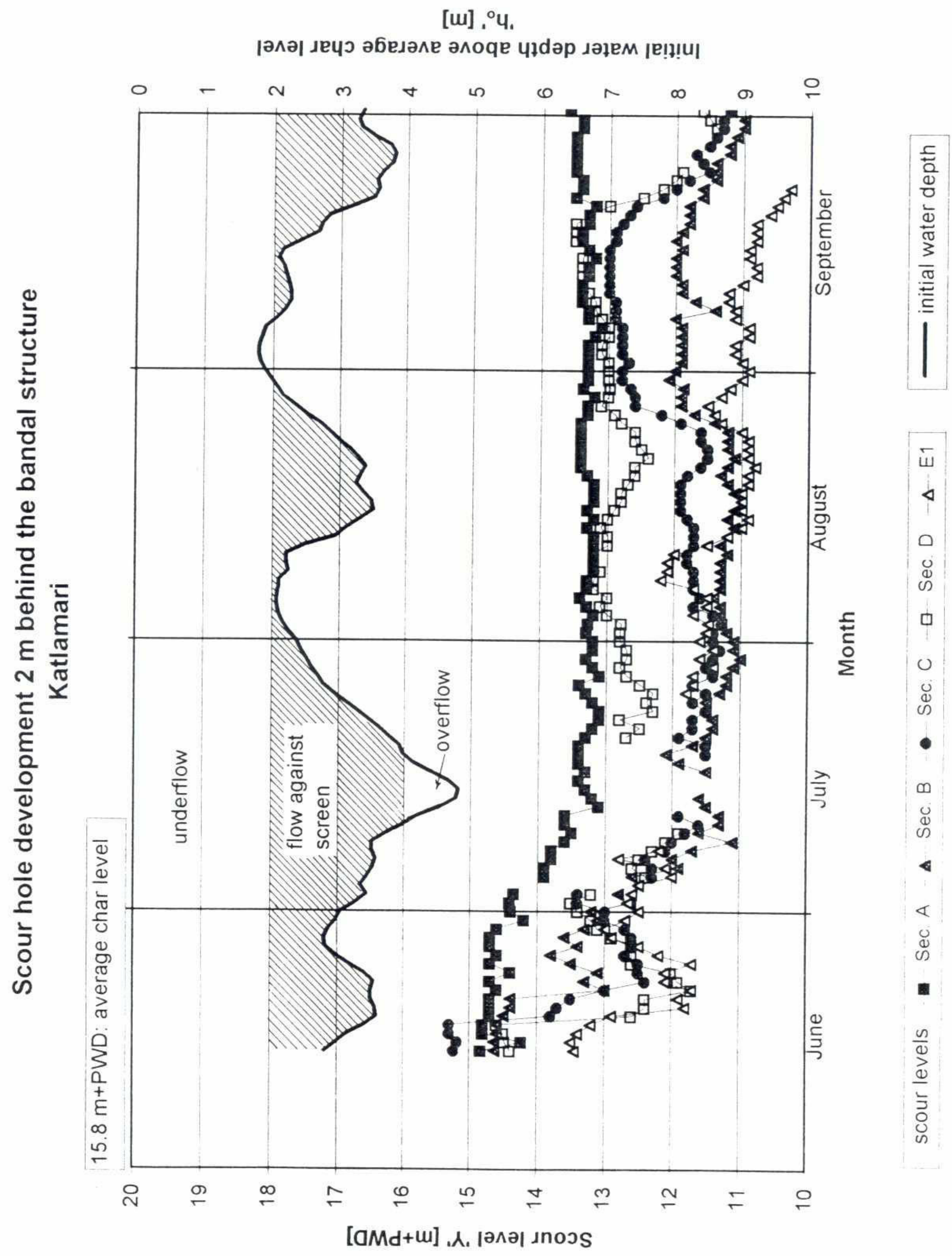


Fig. 6.1: Scour levels at 2m behind the bandals

Notes:

- The location of the maximum depth in the scour hole was varying with time because of the varying water levels and scour depth. However, it is assumed that this depth occurred at 2 m behind the bandals (see also hereafter under "shape of the scour hole").
- The water depth above the average char level has also been plotted in the above figure to show the relation water depth / scour depth (see also Fig. 6.2).
- Fig. 6.1 shows that in the (mainly) rising limb between mid June till mid July the erosion behind all bandals except A had the same tendency. After the steep water level rise in the second week of June, when about 3/4 of the bandal screens were submerged, the scour generated by the screens developed rapidly. At all locations (except Section A) erosion speeds up to 1 m/day and more were measured. After this fast immediate response the scour depth stabilized and followed gradually the variation of the water depth.
- In the second half of July when the water levels were falling till nearly the bottom of the bandal screens the scour depth remained constant or even increased slightly behind Section B. This means that there was hardly any net fill-in and suggests a low sediment transport; Section D deviates in this respect. Another reason of the low fill-in can be the flow component through the scoured channel parallel to the bandals. Behind the bandals the river bed silted up hampering the flow perpendicular to the bandal screens.
- The missing data around mid July was caused by the fact that during the highest water, part of the bandal structure became inaccessible and measurements had to be interrupted.
- The total different development of scour behind bandal Section A is most probably caused by the sheltered position near the high river bank.
- From the point of scour depth there is no clear preference for the design of Section B, C or D. Section D seems better but is less effective because of the shape of the screens.

Scour depth is a decisive parameter in view of the applicability of improved bandals. Too much scour endangers the stability of the structure. Long, safe piles are costly or not available above a certain length. For the improved bandals at Katlamari more than half of the construction costs were for the foundation. So, an improved understanding and predictability of the scouring process is essential for the optimization of the bandal design. Therefore, some further analysis of the scouring process follows hereafter.

(c) Shape of the Scour Hole

Twice a week detailed measurements of part of the scour hole behind the screens were done. Although positioning of the survey boat was often difficult and consequently the accuracy rather low, some results have been plotted in Figure 6.2.

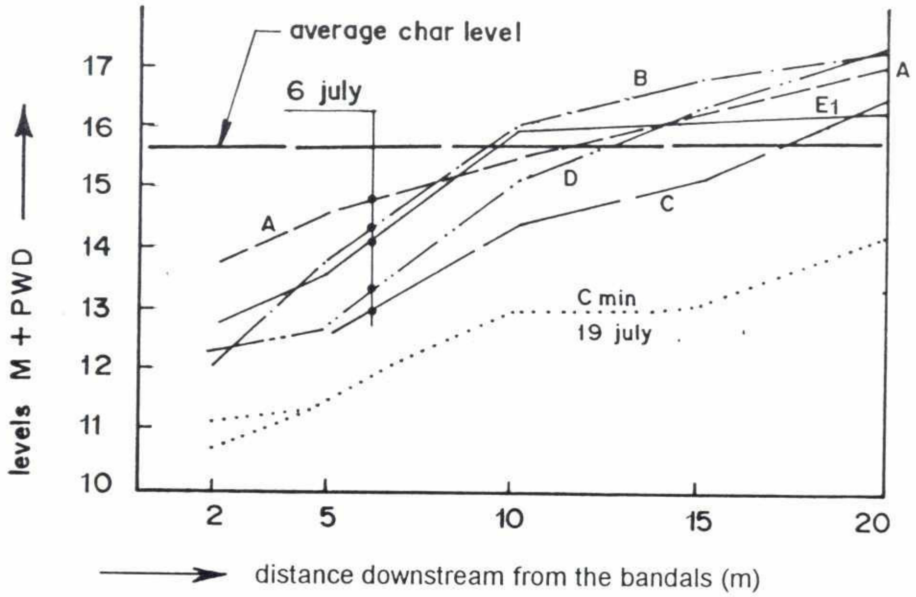


Fig. 6.2: Shape of scour hole downstream from the bandals

Notes:

- the profiles in the middle of the various sections have been plotted
- the profiles are taken perpendicular to the longitudinal axis of the bandal structure, (see LEGEND).

The shape on other dates remained about the same. In the first half of the 1997 monsoon, the deepest scour hole occurred on July 19, hence a couple of days after the high water peak. From the above Figure 6.2 it can be concluded that the maximum scour depth developed near the measuring line at 2m behind the bandals, see also the Figure 6.4 on the shape of the scour hole under the bandals. The length of the downstream part of the scour hole varied in place and time. Mostly the maximum dimensions were found behind bandal Section C. The downstream slope of the hole varied with time and along the bandals.

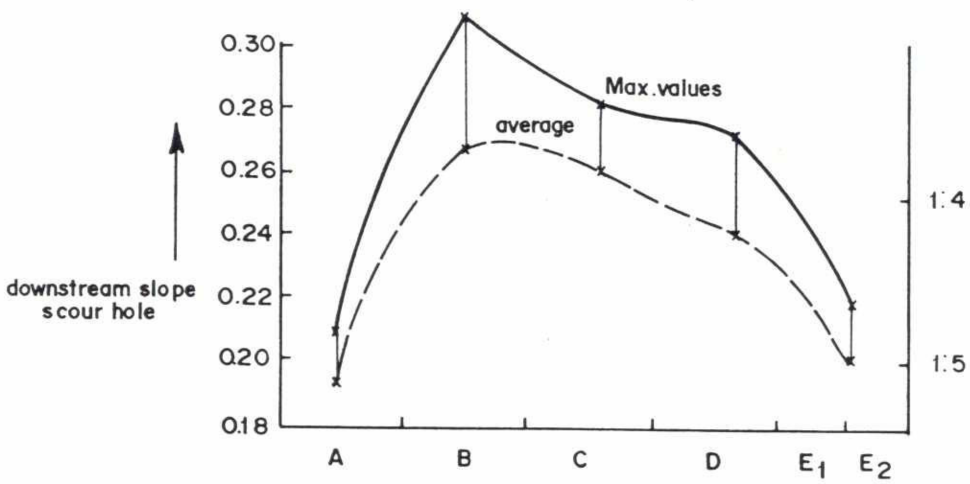


Fig. 6.3: Downstream slope behind the bandals

The length of the scour hole varied with the depth (and the slope) and was usually within 20m. As a maximum 28m (downstream from bandal Section C) was observed. This was dangerous in view of the anchoring system of the ropes of Section C which were fixed at 20m from the bandals at a level of 15 m+PWD. Downstream from the scour hole accretion was observed (see Fig. 6.8).

At the upstream side of the bandals only one very detailed measurement was executed to find the shape of the scour hole upstream and under the bandals. The result of this measurement is presented in Figure 6.4.

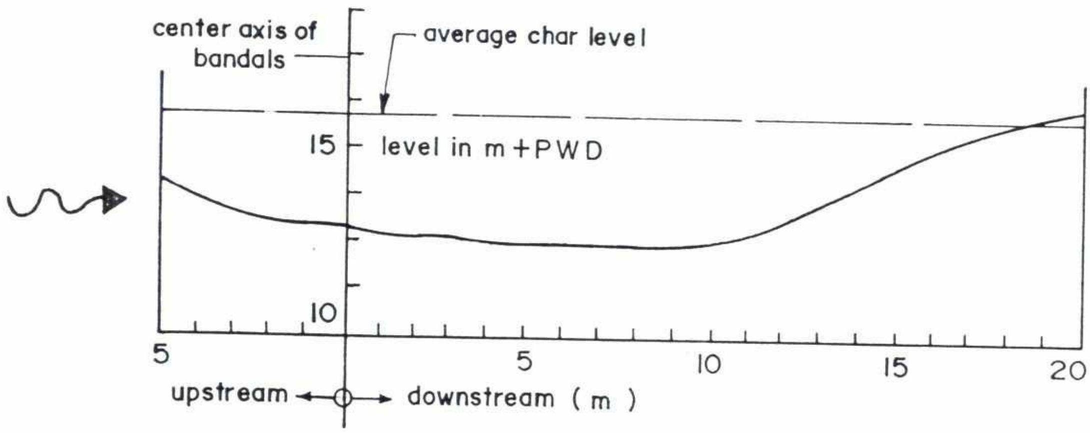


Fig. 6.4: Shape of scour hole under bandal (mid Section C)

From Figure 6.4 a few things can be concluded:

- the location of the maximum scour depth was not very sharply defined, say 4 to 10m downstream from the bandals;
- the upstream slope was about 1:3, and
- the upstream length of the scour hole was about 10m

Assuming that the shape did not change during erosion or siltation the maximum length upstream (during maximum erosion depth of 5.1m) is estimated to be about 30m. Hence, the total maximum length of the scour hole is 40 m, which is 8 times the scour depth. For the design a scour length of 20 to 30m was estimated.

The final shape of the scour hole, as measured in January 1998 is depicted in Fig. 6.5.

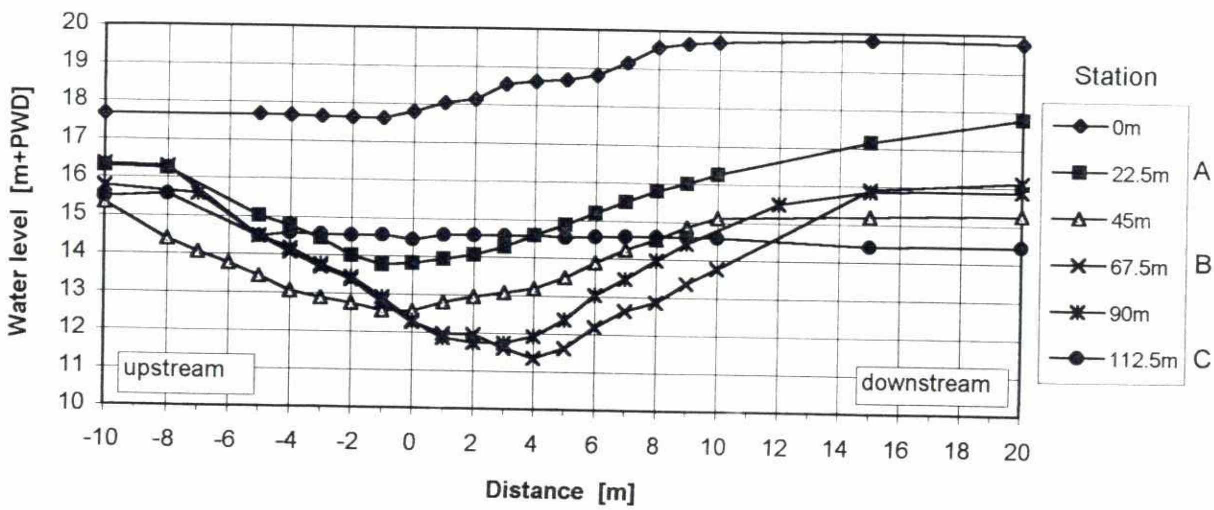


Fig. 6.5: Final shape of scour hole in January 1998

(d) Speed of Scour

The speed of erosion and sedimentation can be seen from the time series of $y(t)$ as plotted in Fig. 6.6. The scour speed, being the changes of scour depth in a unit of time can be calculated as $\frac{dy(t)}{dt}$

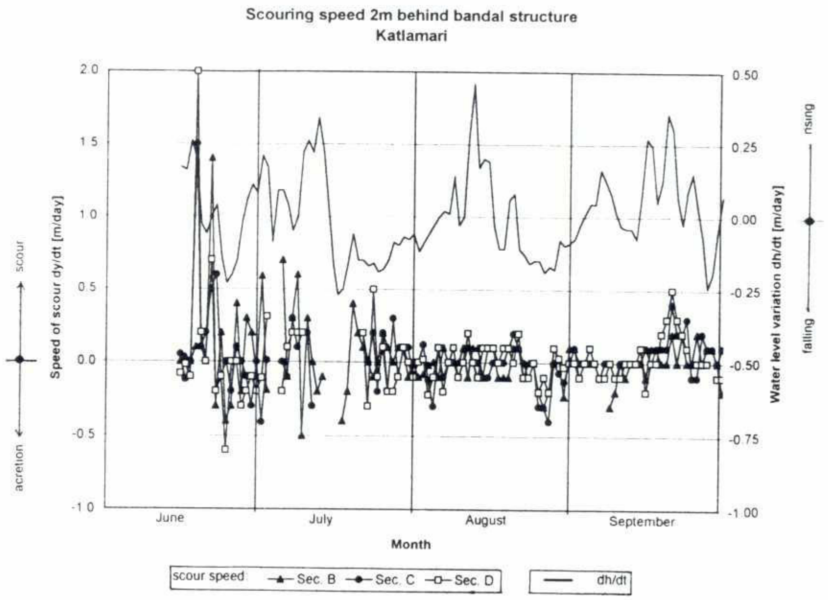


Fig. 6.6: Speed of scour

From the graph the following can be concluded

- the initial scour speed, occurring when the bandals screens were about 3/4 of their height submerged, was considerable. A maximum of 2 m/day has been measured.
- Thereafter scour speeds were moderate <0.5 m/day. After the peak flows of mid July the speed reduced to values ≤ 0.2 m/day (accretion or scouring or just the inaccuracy of the measurements)

(e) Scour Predictions

Part of the design work of improved bandals consisted of scour predictions, which were made by the end of 1996. For details see [11]. Here only the main results are summarized.

As a first step a theory had to be selected as theories of scour below bandals do not exist. Herewith a fully developed velocity profile at the upstream boundary of the scour hole and the expected flow pattern in the scour hole were considered. No satisfying similar cases were found. The most reasonable theory used to assess scour behind long bed protections behind hydraulic structures (sill, sluice, etc.) was then taken. For these cases Breusers (1966, 1967) found for the development of clear water scour under steady flow conditions:

$$\frac{y_m}{h_o} = \left[\frac{t}{t_1}\right]^{\gamma} \qquad (1)$$

where

γ	=	an exponent; often $\gamma = 0.4$ is taken and
t_1	=	the time at which $y_m = h_o$, the so-called time scale of scour

The time scale of scour is usually expressed as [17, 18]

$$t_1 = \frac{K\,\Delta^{1.7}\,.\,h_o^2}{\left(\alpha\,\overline{u} - u_{cr}\right)^{4.3}} \tag{2}$$

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w}$$

where

ρ_s = specific density of sediments
ρ_w = specific density of water

$\overline{u}, h_o$: see Figure 6.7
$\alpha = 1 + \sigma u / \overline{u}$ with $\sigma u / \overline{u}$ = local relative turbulence intensity
u_{cr} = critical flow velocity = f (D_{50}) with D_{50} = medium grain size diameter
K = factor derived from model tests. For the scour behind a bed protection
K = 330 for t_1 in hours

The above parameters have been applied to the improved bandal structure as defined in the following figure:

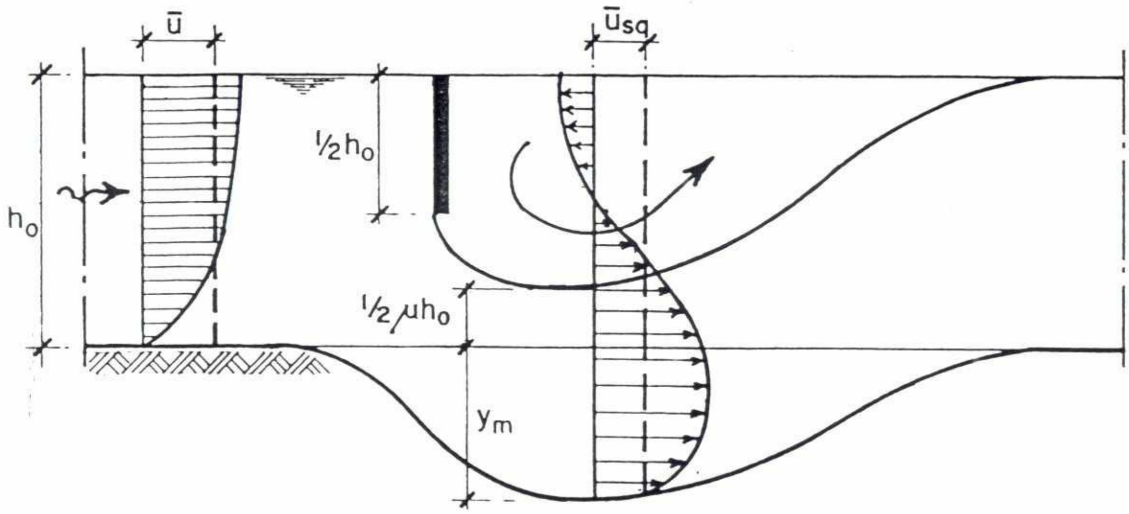


Fig. 6.7: Scour parameters for bandal scour

The scour depth computations of December 1996 were based on the following estimates:

$K = 330$, $\Delta = 1.65$, $h_o = 4\,m$, $\alpha = 1.75$,
$\overline{u} = 1\,m/s$ and $u_{cr} = 0.28\,m/s$ for $D_{50} = 150\,\mu m$ (see sieve analysis test site [41] showing usually values between 150 and 200 μm).

From equation (2) follows t_1 is about 100 days and equation (1) yields $y_m = 3.8$m. For additional scour around piles 0.4m was taken, so a total scour depth of 4.2m was estimated [11] for clear water scour.

From Fig. 6.1 it can be seen that the maximum live-bed scour of the bandals is about 4.8m which is considerably more than the 4.2m clear-water scour estimated before. In one case even a maximum of

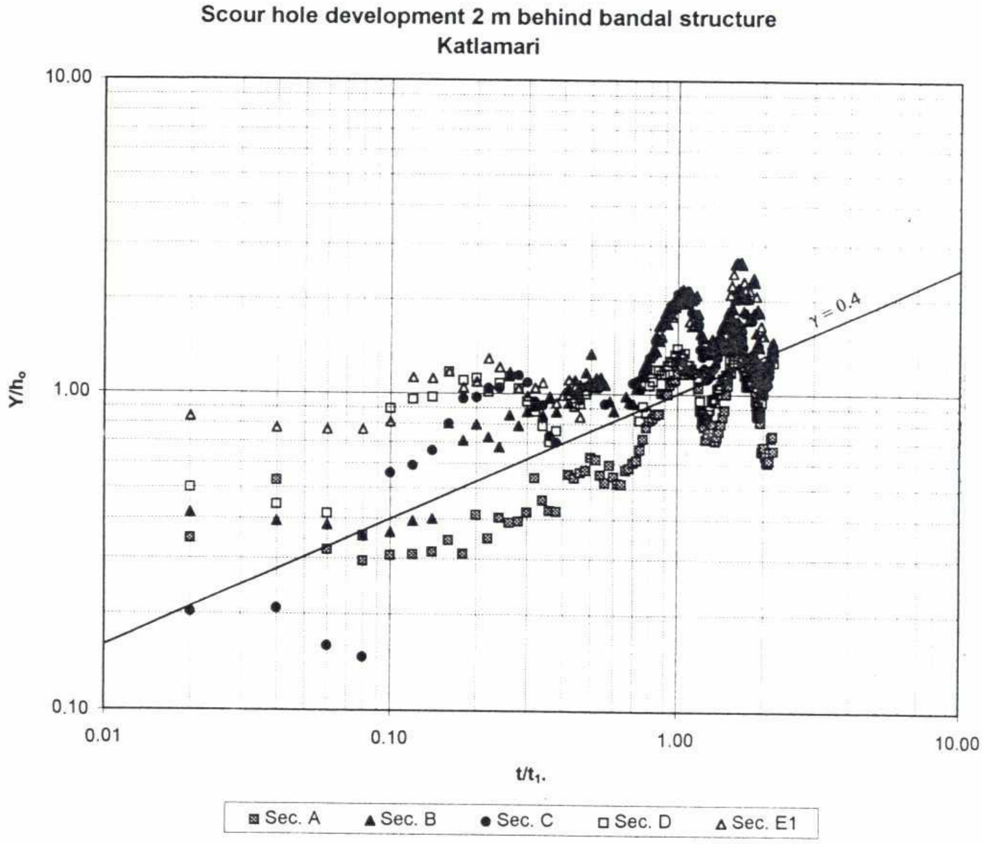
5.1m scour was measured behind the transition of Sections B and C, possibly due to some shifting of the bandals of Section C.

(f) Calibration of the Theory

The applied theory viz. the equations (1) and (2) can always be adjusted to improve the description of the live-bed scour under bandals.

In the empirical formula for the assessment of t_1 (eq.2) the factors K and/or α can be adjusted. More extensive possibilities to adjust also the exponents in the equation are not elaborated in the context of this evaluation report. There is some evidence that K is not sensitive for the upstream roughness [20 p.47]. This means that there is no reason to adjust K values because of the absence of a bed protection upstream of the scour hole.

There are, however, reasons to increase the α value somewhat. The relative turbulence is higher than expected. The detailed velocity measurements in the scour hole were executed as 3 series of 10 seconds per point. The variation between the series is an indication (not a determination) of the turbulence. From these figures relative turbulence values of about 30% seem likely. This leads to α values of 1.9 instead of the estimated 1.75 value. Another estimate comes from earlier model investigations [20p.50] where for a case without sill and without bed protection an α - value of 2.0 is indicated. With this value t_1 becomes 50 days and y_m =$5.1\ m$. This seems perfect, assuming that the $\gamma = 0.4$ value of equation (1) is correct. This has been verified in the following figure.



LEGEND

y	=	the scouring depth at 2 m behind the bandals measured below the average initial char level of 15.8 m+PWD, at time t
h_o	=	the upstream water depth above the average initial char level of 15.8 m+PWD at time t
t	=	time in days after the moment the rising water levels touched the screen on June 11, 1997 at 12:00 hrs.
t_1	=	50 days

Fig. 6.8: Scour depth / water depth ratios versus time

From Figure 6.8 the following can be observed:

- in the first days ($t / t_1 < 0.1$=5 *days)* the y/h_o ratio decreased because there was hardly any scour;
- the low initial y/h_o values of Section C were caused by sand supply after construction of the bandals to rise the river bed to its original level. This was done in Section C only;
- around $t / t_1 = 0.1$ the scour ratio jumped up in all sections except Section A;
- thereafter upto say $t / t_1 = 0.6$ there was an increasing tendency, but at some locations only. Moreover, this increase was not constant and not equal at the various sections. This means that an appropriate γ-value could not be determined (see ideal slope for γ=0.4);
- the variation as a function of time from $t/t_1 > 0.7$ was mainly caused by the water level in combination with a minimum of scour, and
- total conclusion must be that the tested theory appears to be less appropriate for the description of the development of scour under bandals. Only a first rough estimate of maximum scour in the season is possible.

Note: the log-log scales used, transform equation (1) into

$$\gamma = \log\left(y / h_o\right) / \log\left(t / t_1\right) \tag{3}$$

For γ = constant this means a straight line in Figure 6.8.

The main conclusions regarding scour depth are:
- bandal scour starts only after screens are for 2/3 submerged;
- scour was very fast, in only a few days a kind of equilibrium was achieved;
- bandal scour speeds up to 2 m/day have been measured behind Section C because of the initial fill-in, and
- the theory on scour behind sills does not describe the scour development. The theory underestimated the maximum scour depth by 20%.

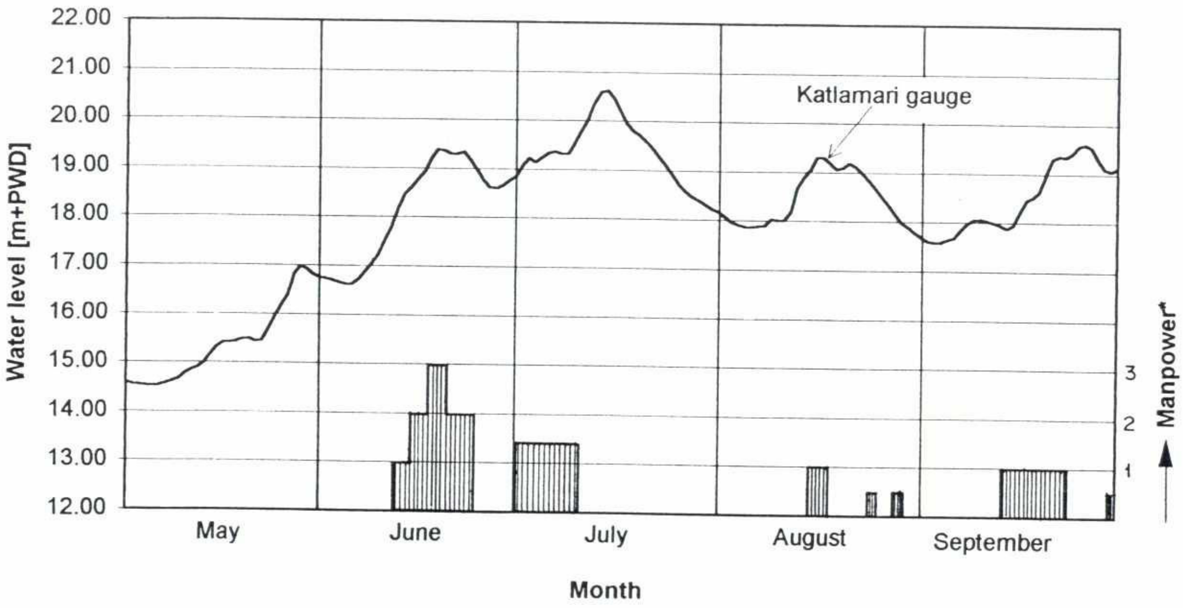
Sor

(g) Effect of Floating Debris

As floating debris like water hyacinths and banana trunks may cause additional scour and/or additional load on certain structures, regular cleaning of the bandal area was executed. From the logbook some relevant remarks are summarized hereafter.

June 11	Start of cleaning from floating debris.
June 12	Cleaning continued.
June 13	A floating wire was installed upstream from the bandals to prevent the debris from hitting the structure. Unfortunately this wire snapped after a few days and was not replaced.
June 14-27	Permanent cleaning was done by a team of about 10 persons with a mechanized country boat. Sometimes a second team was engaged with an extra boat, working also during night time.
June 17-18	A field of floating debris was reported in front of the Sections A, B and C, covering an area of 15 to 25 m with a thickness of 0.2 to 0.3m.
June 20	Debris was observed to be trapped behind the bandals (this confirms the flow pattern as indicated earlier, see for instance Fig. 6.4).
July 01-09	Cleaning was resumed.
July 10-18	The bandals were completely under water and floating debris could pass the structure.
July 19 - August 13	No floating debris observed. During this period (falling stage) the structure remained clean.
August 14-16	About 10 m of floating debris in front of Section C.
August 16-23	No floating debris in front of the structure.
Aug. 24 and 28	Some cleaning done with a few persons only.
Aug. 25-27, Aug. 29-31	No floating debris in front of the bandals.
Sept 08, 10-14	No floating debris in front of the bandals.
Sept 09, 15	Some cleaning done with a few persons only
Sept 17	Floating debris observed in front of Sections A, B, C, D and E1, covering an area of 3 to 10m with a thickness of 0.3 to 0.5m.
Sept 17-19	About 4 persons engaged for cleaning the floating debris each day.
Sept 20	Floating debris observed in front of Sections A, B and C covering an area of 2.5 to 10m with a thickness of 2.5 to 3m. Seven persons and one engine boat engaged for cleaning the floating debris.
Sept 21-27	Cleaning continued.
Sept 28-29	No floating debris in front of the bandals.
Sept 30	Some cleaning done with some persons only.
October to December	No floating debris observed in front of the bandals.

The cleaning of the bandals from floating debris during several periods in the monsoon of 1997 has been depicted in Figure 6.9.



* indication of manpower for cleaning of bandals
scale: 1 = one team of 7 to 10 persons during 12 hrs. per day

Fig. 6.9: Cleaning of bandals

From the figure above and Fig. 6.1 showing the development of the scour hole, it is concluded that there is no important relation between floating debris and scour. Most important is that there is no floating debris during and just after the first rise of the water levels when screens become partly submerged and scour appeared to be maximum.

Yet there are good reasons for cleaning the structure from floating debris, such as:

- piles of floating debris will affect the hydraulic conditions around the screen obscuring the appropriate functioning of the bandals;
- debris will cause additional load on the structure. This will happen particularly on special parts such as the ropes of Section C, and
- the flow hitting the screens under an angle of about 45° will try to push the debris along the bandals. Especially at the irregular parts of the screen surface (for instance the connections between the various bandal sections). This may cause considerable forces in the longitudinal direction of the bandals. The shifting of the screen in Section C parallel to the axis is caused by floating debris pushing the ropes.

It is concluded that cleaning of the structure is important. Although the total period of cleaning appeared to be only about 1 month during the 1997 monsoon, the amount of work appeared to be considerable. The installation of a "floating rope" to divert the flow angle of the debris seems a sound one. More debris will bypass the structure and the remaining part can easier be removed. The strength of the locally made jute rope appeared insufficient. Some tests applying other material for the "floating rope" are recommended.

6.2.3 Structural Aspects

After the monsoon of 1997 it could be concluded that the bandal structure survived the hydraulic conditions (the occurring flow velocities and waves) reasonably good. Inspection of the substructure during low water (January 1998) revealed that the condition of the substructure of Sections A through D was excellent. However, nothing of the piles of Section E was found. In a number of cases the superstructure was slightly damaged. Therefore the relevant observations (as noted down in the logbooks) have been listed below.

Date	Event
~ June 17	Shone (sisal) rope keeping the debris from the bandals broken. Some erosion at the bandal stem (of the high bank).
June 19	Some settling of the various bandal sections observed.
June 21	0.2m opening between Sections B and C due to shifting of screen C probably caused by floating debris.
June 28	Extra bank erosion probably due to waves (stormy weather).
July 06	0.3m opening between Sections B and C.
July 09	Navigation jack broken.
July 18	Repair of gangway.
July 23	Difference between A and D screen levels nearly 0.5m. Opening between B and C increased to 1.2m at the upper side of the screen.
August 26	6 out of 15 screens of bandal C shifted due to storm.
August 29	5 of lowest row of mats (with a height of 0.5m) of Section B were damaged due to wave and current
September 09	Scour level of Section E2 is 10.40 m PWD, hence pile depth is not safe if water level rise
October 07	Bandal Sections E1 and E2 is washed out

Summarizing it can be stated that only minor (at least easy repairable) damages were detected. The troubles with the screens may be related to the different type of mats applied. For the pattern of mats applied see the following table:

Section	Total Length (m)	Length of double screen (m)	Length of single screen (m)	Number of double screen	Number of single screen	Size of each screen
A	45	21	24	7	8	3.0 x 2.41
B	45	21	24	7	8	3.0 x 2.0
C	45	21	24	7	8	3.0 x 2.0
D	45	21	24	7	8	3.0 x 2.0
E1	15	6	9	2	3	3.0 x 1.75
E2	15	6	9	2	3	3.0 x 1.0

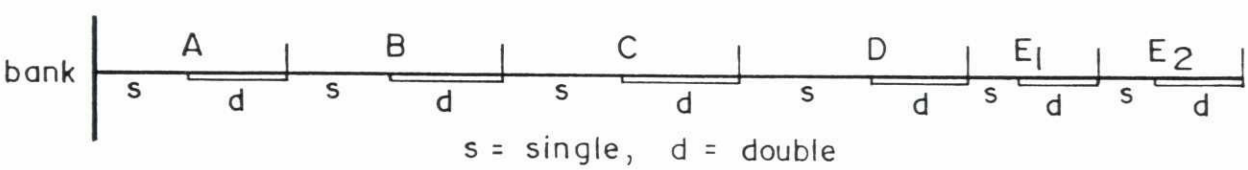


Table 6.1: Pattern of double and single mats in the bandal screens

An inventory, carried out in January 1998, gives some interesting information about the performance of the bandals. In the following the general and the special aspects are briefly described.

- The damage pattern of the mat however was not corresponding with the single/double pattern of Table 6.1. In Section A and C the mat looked significantly better, as the vertical supporting structure could prevent the lower part from being torn off. This has happened in quite some parts of Section B and D. Some of the mats showed holes, either from a weaker structure of the impact of banana trees or other hard floating debris. A second lighter mat, additionally placed on the surface of all sections in downstream direction was rotten at the bottom. This is due to the longer time in water. In general this doubled screen looked much denser.

- The wire bindings of all sections looked terrible. The steel was totally corroded and cracked under slight loads. This means that even though the bamboo showed no signs of deterioration, that all bindings have to be repeated after one season. The same is with the metal sheets, bolts and rods, for the pile head connection. All were extremely corroded. They could survive simply because they are thicker than the wire used.

Some peculiarities of the single sections are noted below:

- In Section A the structure seemed unchanged compared to it appearance before the flood. However, the screen had settled, i.e. is pushed down from the hydraulic load, especially by the vertical component of the weight. This could be seen at the upper diagonal, which was bended down over all of the length of the structure. Secondly the steel rod from the first pile row to the back, installed for inducing tension forces from the back of the structure into the first piles, was quite often displaced or missing. The main problem was certainly due to corrosion, however, part of it can be to movements of the supporting bamboo poles.

- In Section B the lack of the tension diagonal was especially noteworth. The structure had survived with most of these diagonals missing or loose. However, due to the little resistance of the wire bindings it is not clear, if the diagonals got lost during the last peak, when the wire was already corroded, or whether the binding was not strong enough for the forces. The survival of the structure without these diagonals, makes the first solution more probable. In the end for further developed structures these bindings must be improved.

- Section C was bended in downstream direction. Due to forces along the screen and especially the increased force from the floating debris that was caught in the upstream ropes, the structure was bended about 1 meter in downstream direction. This behaviour was expected and consequentially diagonals to carry this load were foreseen. Even one vertical pile (consisting of three bamboos) was broken due to the deformation. But this was the only one, i.e. the rest could perfectly follow the deformation. However, all diagonals were buckled or missing due to overloads. This must be changed. The overall performance of the rope anchors was very well. No unusual deformations appeared due to uneven elongation of the ropes. And the main loads could be transferred out of the area of the scour hole.

- Section D did not perform that well. The main problem was the special screen design. This could not carry the loads without major deformation. The substructure and the mixed pile foundation showed no signs of deterioration, thus allowing the conclusion that bamboo and bullah piles can be used together. In some parts bamboo dowels were used instead of hardwood pegs. Both showed no difference in their capacity to carry loads. No type led to unusual splitting of the bamboo or breads. Consequentially, the cheaper and easier to build solution can be recommended for further structures.

- As Section E2 was washed away early September 1997 and Section E1 during the last peak end of September, only the four main Sections A to D can be analysed in detail. Section E, however, can indirectly be evaluated. The structures itself withstood the flood until the pile foundation was no longer able to carry the hydraulic load. At that time only about 2 meters of pile were embedded, then the bandal was washed out. When the channel had very low water levels, nothing of the old pile foundation could be found. This was a clear indication, that the connection between piles and superstructure was very good. If the superstructure had failed, at least part of the pile foundation would have been visible. In this section only vertical piles were used. It is not clear, but rather probable, that inclined piles would have withstood the loads better, as they can better cope with horizontal loads. No final answer can be given, as it was from the beginning clear and was shown, that once a deep channel approaches the structure the dimensions of the natural and low cost elements are not sufficient to withstand this major attack.

  Looking back and from the experience gained it is reasonable to use slightly backwards inclined screens with an angle less than in Section A. The bindings must be improved, i.e. important bindings for tension should be doubled. For a second season it is advisable to renew all wire bindings, even though he bamboo seems to be strong enough to withstand the forces. Section C is still the most flexible and best solution even though the design principle is not suitable for moveable screens. Consequentially Section B - using good elements from other sections - will further be developed.

6.2.4 Hydraulic Loads

For the design of the bandals the hydraulic loads viz. the forces due to currents and waves were estimated. This was done first for a rigid bandal structure [11]. During the nearly simultaneous design and construction process it became clear that considerable reductions especially of the design wave load were justified if the screens are made sufficiently flexible [9, 10]. In this subsection these design considerations are compared with the actual conditions of the 1997 monsoon. Particularly the following aspects have been discussed:

- discharges;
- flow velocities;
- wave heights, and
- hydraulic loads

(a) Discharges

From FAP 24 measurements in 1995 [25] it was estimated that the discharge through the Fulchari channel was about 15000 m^3/s when the water level in Katlamari reach the top of the bandal screens (9.8 m+PWD) [11], see also the stage-discharge curve in Chapter 5.

The discharges as measurements in the 1997 monsoon are plotted in Figure 6.10.

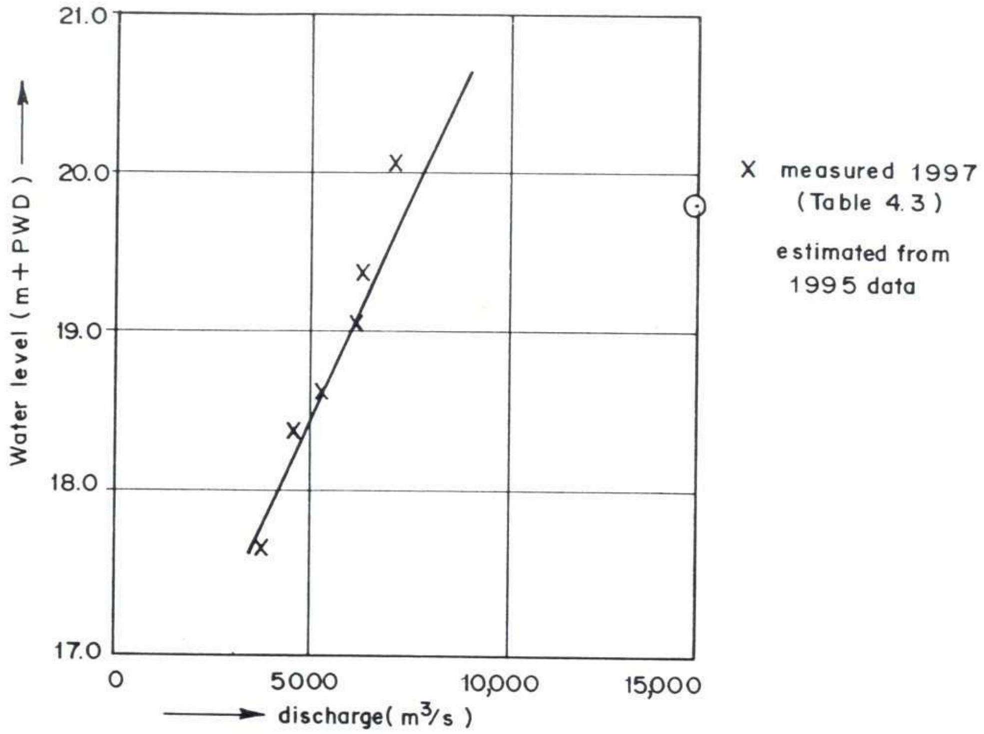


Fig. 6.10: Stage discharge in the Fulchari channel

Similarly for the Katlamari channel the following graph could be made.

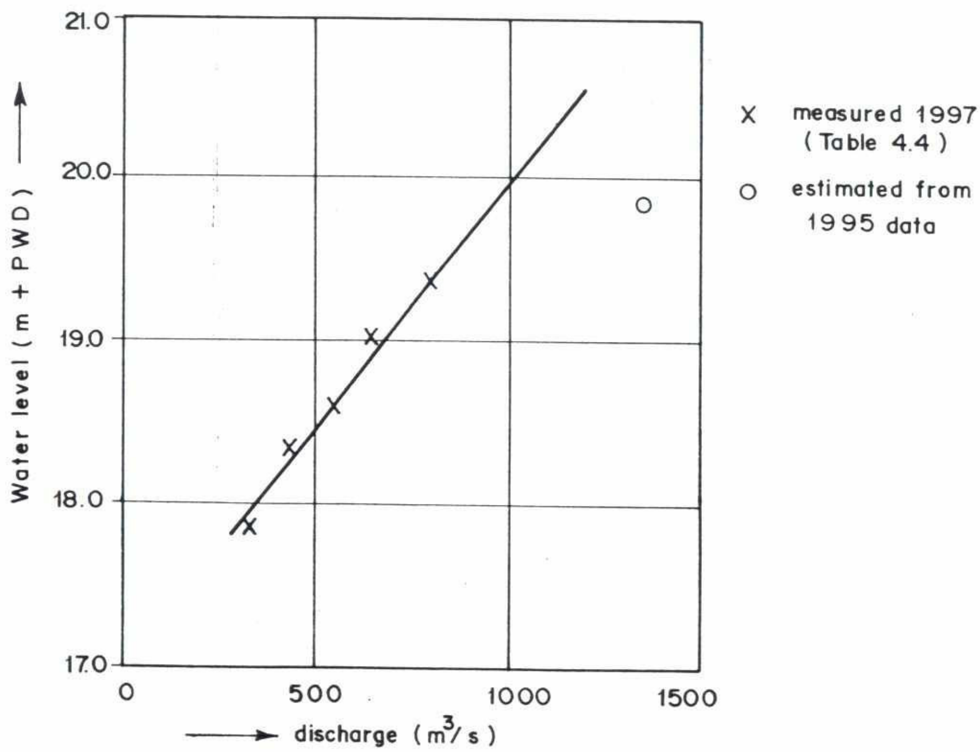


Fig. 6.11: Stage discharge in the Katlamari channel

From the Figures 6.10 and 6.11 it can be concluded that the stage-discharge curve changed considerably between 1995 and 1997. However, the main changes were caused by considerable accretion in the Fulchari channel (especially at the entrance). As a consequence, the wet cross-section decreased, by which the discharges reduced without appreciable changes in the flow velocities.

(b) Flow Velocities
For the hydraulic load on the bandals the flow velocities over the char are important. Earlier estimates varied from 1 to 1.1 m/s [1].

The measurements show an important horizontal gradient over the upstream char area with low velocities near the high bank of 0.2 to 0.3 m/s, increasing to 1.1 to 1.2 m/s near the edge of the deep channel. Also float tracking data confirmed that these velocities occur, for instance near the tip of the bandals. It can be concluded that the estimates of the design velocities were rather accurate.

(c) Wave Heights
Wave heights have been measured at three locations from staff gauges, daily at 8:00, 13:00 and 17:00 hrs. For the data see Annex B.5. The main conclusions are

- the height were relatively small. Not more than 0.2 m was observed at the staff gauges;
- this contradicts with logbook remarks about stormy weather and waves damaging the screen mats, and
- apparently the wave observations did not serve the purpose.

(d) Flow Induced Load
As flow induced loads were not measured, only a few parameters in the load-formula can be considered.

$$F_e = (\rho\, g\, \Delta\, h\, h_s + 0.5\, \rho\, C_D\, h_s\, v^2 \sin\alpha)\, L_s \qquad (4)$$

where

h_s,	=	height of the screen
L_s	=	length of the screen
Δh	=	head (water level difference) over the screen
v	=	flow velocity over the char

The Δh was not measured. It was noticed that during the higher stages, when the screens were overtopped the head was reducing. The maximum Δh, occurring when water levels were close to the screen top at 19.8 m+PWD, was estimated to be 0.07 m. This is considered a fair estimate, although reductions can be applied near to the high bank.

The same holds for the flow velocity v. This means that for the load the F_c / L_s value is to be considered as a maximum force occurring near the centre of Section D.

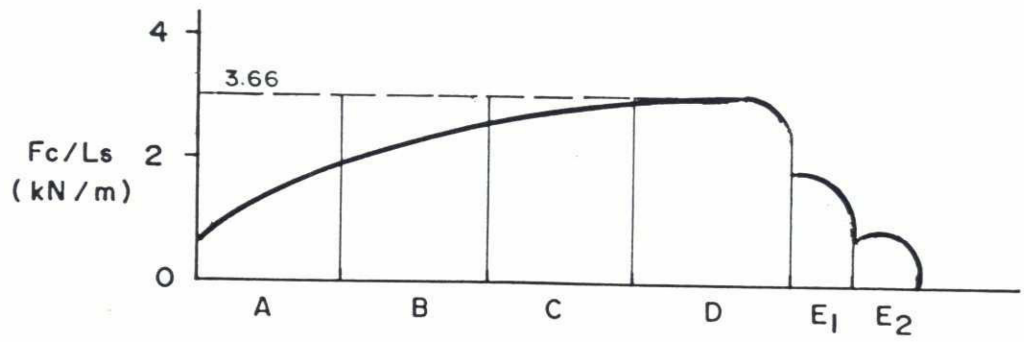


Fig. 6.12: Horizontal distribution of flow induced load

The distribution indicates in Fig. 6.12 that as soon as a more detailed flow pattern is available, load reduction can be applied. Because of the flexibility of the structure the extra 50% load due to dynamic effects can be reduced till say 20%.

(e) Wave load

The background of previous wave load computations [11] is summarized hereafter. A wave height of 0.5 m (and L = 28.9 m) was assumed in the Fulchari channel. Due to lower flow velocities and less water depth the wave height increased to about 0.6 m on the char. This wave with a length of 18 m and a period of 3s ran perpendicular into the bandal screen. According to Goda's theory [26p.115] this wave gives a load of 7.3 kN/m. If the wave of the Fulchari channel of 0.5 m is a significant wave and not a maximum one, the maximum wave height follows from H_{max} = 1.8 H_s = 0.90 m. The load becomes then 15.3 kN/m. This value was the basic load for the design. In view of the flexibility of the structure some wave load reductions were applied [9].

From this approach of determination of wave load some questions emerge about:
i) the applicability of Goda's theory;
ii) the size of the design wave, and
iii) the magnitude of reductions in view of the flexibility of the structure.

i) Applicability of theory

Goda's theory holds for non-breaking and breaking waves and is used in Japanese standard for the assessment of wave loads on closed vertical walls [26, 27]. The theory is developed for a vertical wall on a sill. In case of the bandals Goda's theory was applied as follows.

$$\eta = 0.75\left(1 + \cos\beta\right) H \tag{5}$$

where

β = horizontal angle between wave crest and front wall

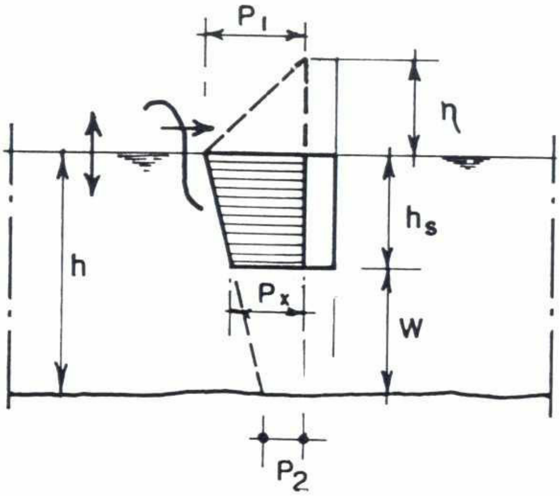


Fig. 6.13: Definition of parameters of Goda's theory

For the absence of a sill Goda's theory simplifies to:

$$p_1 = 0.5\left(1+\cos\beta\right)\alpha_1 \,.\, \rho \,.\, g \,.\, H \tag{6}$$

$$p_2 = p_1 \,/\cosh kh \tag{7}$$

where

α_1	=	$0.6 + 0.5\ (2kh\,/\,\sinh 2kh)$
h	=	water depth (see Fig. 6.13)
k	=	wave number = $2\ \pi\,/\,L$

For perpendicular wave attack $\beta = 0$ and the equations (5) and (6) become:

$$\eta = 1.5\,H \tag{8}$$

$$p_1 = \alpha_1 \,.\, \rho \,.\, g \,.\, H \tag{9}$$

The wave load follows from the assumed pressure distribution (see Figure 6.13). The water levels are at the top of the screen, so $h = h_s + w$, then the load becomes:

$$F = 0.5\left(p_1 + p_x\right)\,\left(w-h\right) \tag{10}$$

$$p_x = p_2 + \left(p_1 - p_2\right)\; w\,/\,h \tag{11}$$

For $h = 4\ m$ and $w = 2\ m$

$$F = p_1\left(1.5 + 1/\left(2\cosh kh\right)\right) \tag{12}$$

The formulae show that the load is linear with the wave height. For the wave height the maximum wave height should be taken. So with the water levels lower than the top of the screen the maximum load occurs.

Weak point in the theory is obviously that it holds for closed walls, whereas the opening under the screen affects the shape of the pressure distributions. For better estimates wave flume tests are probably needed.

ii) Design wave height
Appropriate statistics on wind and waves in the project area (or nearby the project area) are lacking, which makes it difficult to choose the right wave height. To assess possible wave heights in the Fulchari channel the wind and wave measurements in the monsoon of 1997 are considered and some data had been generated using a simplified wave prediction model (Annex G). Results are summarized hereafter.

	Design	Measurement 97	Calculated Annex G	Remarks
H_s	0.5	-	0.3	1)
H_{max}	0.9	0.2	0.54	2)
α	81°	90° - 135°	25° - 55°	3)

Table 6.2: Waves in Fulchari channel

Remarks
1) the measured wave height is too low (see also remarks on page 6-18);
2) the computed wave height shows limited heights because of limited wind speeds along the direction of the fetch. Moreover, the used statistics are based on recordings over one season only;
3) in spite of all these incertainties there is certainly scope for reduction of the design wave height till say H_{max} = 0.7 m or even 0.6 m.

iii) Dynamics of screens
It seems logical to apply wave load reductions because of the flexibility of screens. One may consider different types of flexibility such as:
- flexibility of the mats;
- 3-dimensionality of flexibility and load, and
- 2-dimensional motion of (rigid) screens

In view of the design of the bandal Section C and D, the last point is very relevant. The way these bandals can swing is rather different

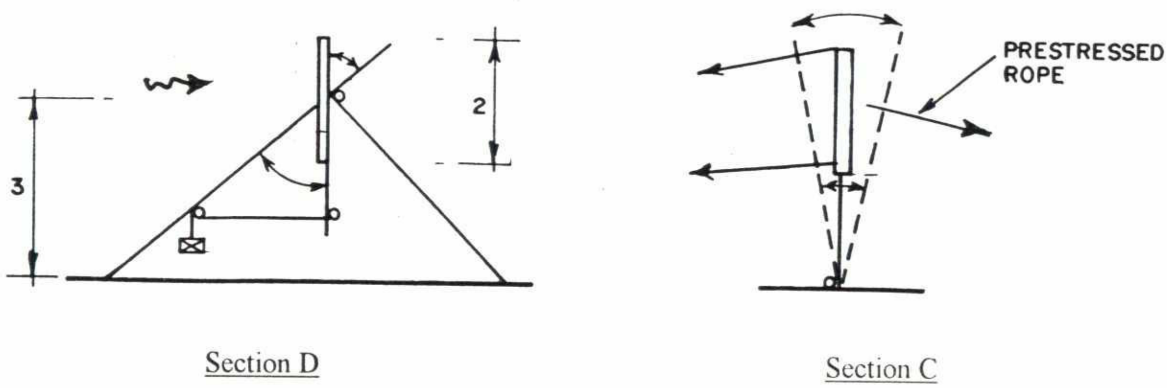


Fig. 6.14: Swinging of bandals C and D

The swinging amplitude of the bandals in Section C is very much depending on the prestress and the elasticity of the ropes. The way of swinging of bandal C follows, at least partly, the orbital motion of the wave and is from that point of view preferable to the way of swinging of bandals in Section D. Dynamic computations are needed to find the relation between the flexibility and the appropriate load reductions. One of the problems with this type of computation is the assessment of the so-called "added mass" (water body which moves with the bandals). Dynamic computations have not been executed in the context of this evaluation report.

6.3 EARTH DAM

6.3.1 General Behaviour

The structure of the earth dam appeared to be strong. The main dam body consisting of sand and protected by Hazen and gunny bags was highly stable in the currents.
Initially, the dam crest was covered with a layer of 0.2 m of sand to protect the dam crest against damage. In June, however, this protecting layer appeared to be eroded and some damage of the gunny bags was noticed, especially along the upstream water line, probably caused by cattles. Due to moderate wave action the damaged bags were partly emptied and bags started shifting. From this it is concluded that the positioning of the bags need to be changed. The positioning was based upon the idea to lay the bags roof-tile wise in the current (see sketch).

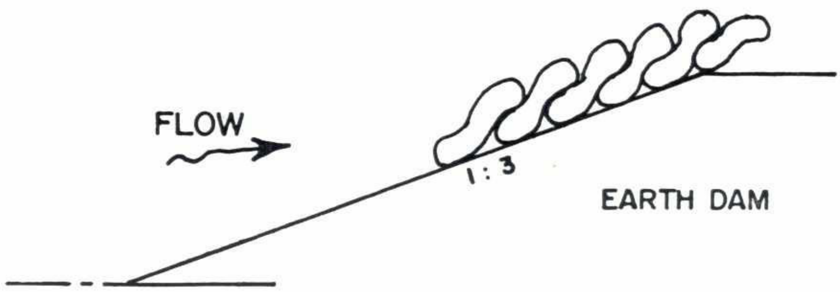


Fig. 6.15: Details earth dam

A horizontal positioning of the bags with the length in the flow direction will probably be more stable.

During the construction of the dam no special measures were taken to compact the sand body. As a consequence the dam started settling during the rising waters. Before the dam was overtopped, measurements showed that the dam crest level was about 0.1 m too low. This was corrected together with the above mentioned moderate damage by replacing the upper layer of the dam crest.

The bed protection length at the downstream toe of the dam was kept very limited as relatively small discharges over the crest and considerable downstream channel depth (upto 7.5 m below the crest) resulted in very moderate attack. The channel bed protection appeared to be in good condition after the 1997 monsoon.

Problems with the dam occurred at both ends. In fact the connections between the dam and the banks of the Katlamari channel was the weakest shackle in the chain. At both ends erosion took place. Very fast at the south-eastern end of the dam near the guiding dam and slowly at the other end near the high bank. The slow erosion could be stopped during the monsoon by closing the erosion gap with a cofferdam. The fast erosion was not counteracted. From the observations and measurements a detailed picture of the hydromorphological processes around the dam has been obtained. Detailed analyses of these processes lead towards insight into possible improvements of the design.

6.3.2 Erosion of Dam

The erosion at the south-eastern end of the dam was very fast as can be seen from the following description of the main events.

June 10, 1997 23:45
The water levels continued their sharp rise of the last days (of about 0.2 to 0.3 m/day) overtopping the flattened char area adjacent to the dam. At that moment the head over the dam was 0.3 m. So sheet flow occurred (as usual when a char is overtopped).

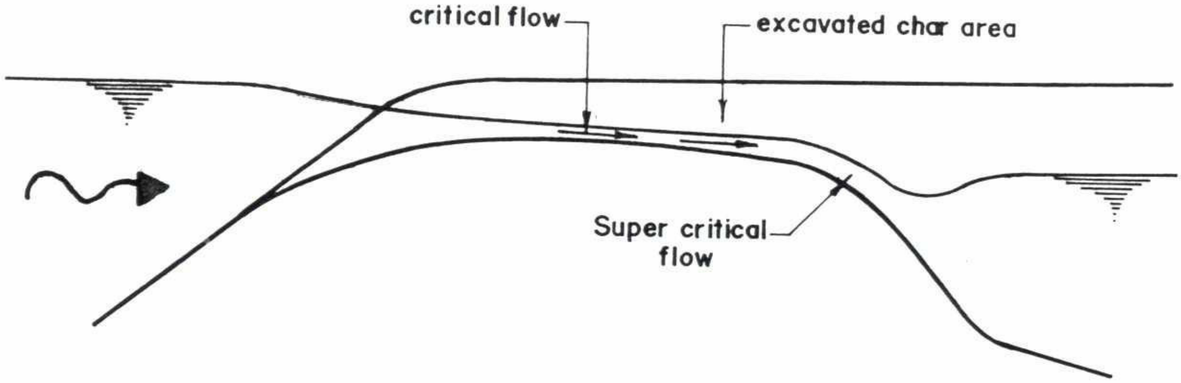


Fig. 6.16: Sheet flow overtopping char

The flow velocities on the char were rapidly increasing, becoming much higher than the critical flow velocities (say 0.4 m/s), which means, that erosion started soon after overtopping. A new channel bypassing the dam developed rapidly. This means in fact that the light sill of two layers of gunny bags had no effect.

June 11, 1997 early morning
During the night hours the erosion of the char was as fast as in the morning at 08:25 hrs. It was observed that the guiding dam had been completely eroded. This means that the light guiding dam, constructed on the slope of the left bank of the Katlamari channel was simply washed away by the new developing channel.

June 11, 1997 10:00
The south-eastern tip of the dam (about 7 m) was damaged.

June 11, 1997 13:30
Already 19 m of the main dam was damaged.

June 12, 1997 early morning
The water levels reached the level of the dam crest and overflow started. Erosion at the dam tip was still strong. The flow through the bypass was already considerable because the initial head over the dam of 0.3 m had been reduced to less than 0.2 m.

June 12, 1997 16:30
About 35 m of the main dam had been eroded. So 85 m remained. The erosion had been decreased in the course of the day and was nearly stopped in the afternoon.

June 14, 1997 noon
Soundings downstream and parallel to the dam revealed that no traces of the guiding dam could be found anymore. The dam tip at 85 m seemed rather stable. Just in front of the tip, slightly downstreams,

a considerable scouring hole was developing with a maximum depth of 10.4 m+PWD, which means 7.6 m below the dam crest.

June 15, 1997 noon

A sounding track in the flow direction, just passing the tip of the dam revealed that part of the damaged tip of the dam was still existing. Scouring was still on-going as behind the dam tip a deepening of 1.0 to 1.5 m was measured over the last 24 hrs.

July 03, 1997

A detailed survey had been executed with tracks crossing the dam, from which the following profiles were derived.

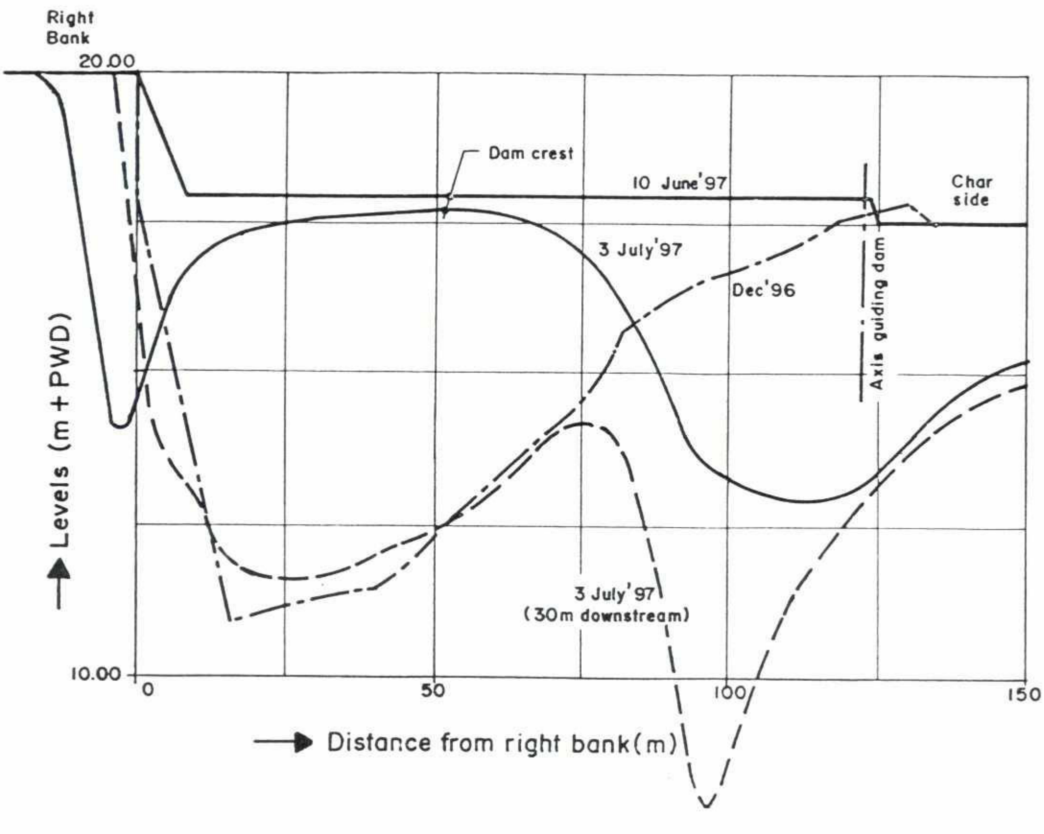


Fig. 6.17: Cross-sections dam site

The figure also includes the cross-section of the Katlamari channel before starting the construction in December 1996 and the levels after construction (June 10, 1997). The figure shows that the main body of the dam, closing the deeper parts of the channel, is still in place. Only some settlement occurred by which the dam crest lowered 0.2 to 0.5 m. At the right bank the dam was starting to be disconnected by erosion. The shallow part of the main dam and the guiding dam disappeared and at that location the new bypassing channel had fully developed. This channel was curling around the main dam, which acted as a kind of groyne. About 30 m downstream from the groyne head the deepest scour hole (8.00 m+PWD) was located. Therefore, a cross-section through the scour hole, parallel to the dam has been added to the previous figure. This cross-section clearly shows that the differences in the main part of the Katlamari channel were small (compare December 1996 section with the July 03, 1997 section at 30 m).

July 15, 1997
The detailed bathymetric surveys showed that the morphological situation around the dam was stabilizing in the first half of July. The deep scour hole near the dam edge remained in the same place but tended to become slightly less deep (from 8.0 to 9.0 m+PWD)

July 25, August 10 and 14, August 19, 1997
Consecutive surveys showed hardly no changes around the dam.

6.3.3 Flow Velocities over the Dam

When the rising water started overtopping the dam supercritical flow occurred on the downstream slope of the dam. On the dam crest the flow was critical and this continued until the water levels were close to three times the total head over the Katlamari channel ($H \rightarrow 3 * \Delta H$, see next figure). Further rise of the water level caused subcritical flow over the dam. For the critical flow conditions see next figure.

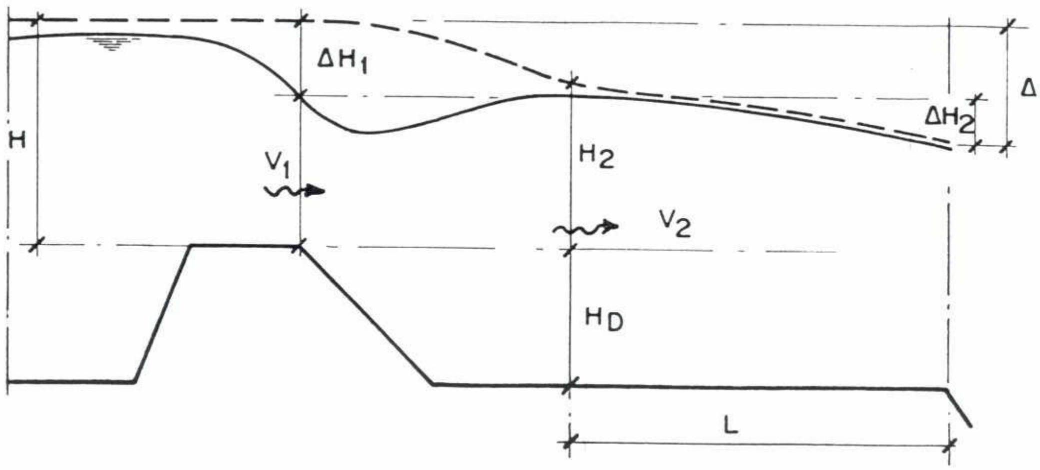


Fig. 6.18: Critical flow over earth dam

The various equations describing the velocities over the dam and in the Katlamari channel for the different types of flow have been summarized in the following table.

Flow Condition	Dam		Channel		
Super critical $H_2 < 2/3\ H$	$\Delta H_1 = 1/3\ H$				$H_2 = H - \Delta H + \Delta H_2$
Critical $H_2 = 2/3\ H$	$\Delta H_1 = 1/3\ H$	$V_1 = \sqrt{2\, g\, \Delta H_1}$	$V_2 = \dfrac{H - \Delta H}{H_2 + H_D} . V_1$	$\Delta H_2 = \dfrac{L V_2^2}{c^2\left(H + H_D - \frac{1}{2}\Delta H\right)}$	$\Delta H_1 + \Delta H_2 = \Delta H$
Sub critical $H_2 > 2/3\ H$	$\Delta H_1 = H - H_2$				$\Delta H_1 + \Delta H_2 = \Delta H$

Table 6.3: Equations to assess dam flow

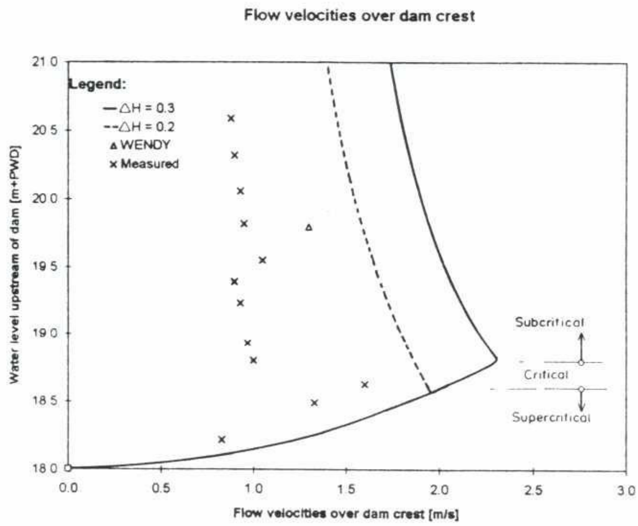
Some notes regarding these equations are:

- the value for v_2 is derived via a simplified continuity equation;
- in the assessment of the energy loss in the channel (the formula for ΔH_2), the hydraulic radius is estimated to be $H + H_D - \frac{1}{2}\Delta H$. This simplification promotes an analytical solution;
- as H_2 values are not timely known, some iteration is needed both for supercritical and subcritical flow conditions.

This set of equations was used to calculate the flow velocity over the earth dam for the following conditions:

ΔH = *0.3 m or 0.2 m*
L = *3200 m*
C = *40* $m^{1/2}/s$

The results are presented in the following figure.



→ flow velocity over dam crest (m/s)

Fig. 6.19: Flow velocities over dam crest

The figure shows that the velocities were increasing when water levels were rising from 18.0 m+PWD (being the dam crest level) till 18.8 m+PWD. At that moment the head over the dam (ΔH_1) was maximum (about 90% of the total head ΔH), while the velocities and consequently the head losses (ΔH_2) in the channel were still small (about 10% of the total head). With increasing water levels the discharge increased and therefore the velocities and head losses in the Katlamari channel increased. This implies that from the total head (ΔH) less was available for the dam and the crest velocities were decreasing.

The head over the dam had been measured several time as noted down in the survey log book. After the dam was overtopped instead of the head, the velocities above the crest were measured. The results have been collected in the following table.

Date	Time	Water Level	Δ h	v
June 09	14:00	17.26	0.30	-
June 10	13:30	17.54	0.29	-
June 11	17:00	17.88	0.20	-
June 12	15:45	18.22	-	0.83
June 13	14:05	18.49	-	1.33
June 14	16:00	18.63	-	1.60
June 15	15:15	18.80	-	1.00
June 16	12:00	18.93	-	0.97
June 17	15:10	19.23	-	0.93
June 18	15:10	19.39	-	0.90

Table 6.4: Water levels and velocities near the dam

Notes :

- After June 18, 1997 the velocities did not change much and remained for a long time around 0.9 m/s (even in mid July with maximum water levels upto 20.6 m+PWD);
- The water levels in the table are from the gauge at the Katlamari test site. The temporary gauge upstream from the dam, used for the head measurements showed nearly the same value;
- The measured combination of water level and flow velocities have been plotted in the previous graph;
- The graph showed that after overtopping of the dam the velocities were lower than expected. This was partly caused by the flow through the bypass which was reducing the head over the dam. Also some uncertainties about the crest level may play a role;
- When the water was rising to about the 18.7 m+PWD level the velocities suddenly dropped to say half of the expected value. Overtopping of major parts of the char may have caused a further reduction of the head and consequently of the flow velocities;
- It should be noted that the theory is based on unit of width calculations which fail above levels with sudden increasing flow width;
- The WENDY computations show smaller velocities than the curves but higher than what has been measured. The model includes the widening of the profile at the char level, but did not include the bypass. Moreover, it should be noted that the WENDY computations show width averaged velocities which are lower than the maximum velocities.

6.4 FLOATING SCREENS

6.4.1 General Behaviour

In June 1997 a floating screen was constructed hanging between country boats. The boats were as rigid as possible connected with each other by means of a horizontal bullah frame. The frame divided the screen in segments with openings in-between. The segments could reasonably well be rised and lowered. Irregular screen handling generated torsion in the system causing troubles for the country boats.

Handling of the system was rather time consuming in the absence of a proper tug boat. But it could be done.

The critical component of the floating screen concept was as expected the anchoring system. A special anchoring system was tested and appeared promising. The behaviour was such that from these qualitative tests was concluded that

- further test are needed, but in more quantitatively way;
- this holds especially for the further development of the anchoring system, and
- the country boats need to be replaced by a special floating system to be designed and constructed for the purpose.

The main events regarding the tests with floating screens are listed below (partly from logbook notes).

Date	Events
June 09	2 country boats for floating screen tests arrived at site.
June 10-20	3-6 persons engaged in making a bullah frame to tie the boats rigidly together and to make the screens.
June 21	Boats were ready for the first tests
July 04-06	First tests at location #1 with boat under 40° and screens lowered in steps from 1.3 to 2.2m draught. Then the system shifted.
July 07-12	Making anchoring systems. Problems with floating debris.
July 13 to August 17	Systematical tests at location #2 with screens. Measuring scour depths, velocities and boat inclination.
July 16-26	Tests at location #3 with a second screen (in-between boats).
July 29-30	The second screen moved to location #4. Placing was difficult. With screen lowered to 2.5m the system shifted.
August 03-17	Measurements at location #4 with the second screen.
August 19-24	Dismantling boats and screens. Recovering of 10 out of 15 anchoring systems.

6.4.2 Local Scour

The main purpose of the tests was to observe the behaviour of the floating screens in the flow. The screens were lowered stepwise up to a maximum draught of about 6 to 8 m, this means that only part of the tests were of interest regarding scour. However, tests with considerable blockage lasted not long enough because of anchor shifting. The scour data showed that usually scour was in the order of a few dm only. In some cases up to 0.5 m was noticed.

6.4.3 Hydraulic Conditions

The floating screen tests were executed under varying flow conditions. During the tests between mid July and mid August the velocities at location #2 were decreasing from 1.7 to 0.8 m/s only.

This means that the load on the screen had been reduced to only 25% of the initial load.

The main conclusion from this observation is that the tests (especially in August) were done in mild conditions. This implies that the impressions about possible holding power of the anchoring system should be considered with caution.

7 CONCLUSIONS

7.1 INTRODUCTION

In the previous two chapters the impact of the measures on the river (Chapter 5) and the behaviour of the structures in the 1997 monsoon conditions (Chapter 6) have been discussed in detail. In this chapter the conclusions are drawn and the positive and the negative experiences gained during the 1997 monsoon have been listed per structure (Section 7.2). In Section 7.3 suggestions are given for further improvements of the various structures.

7.2 EXPERIENCES

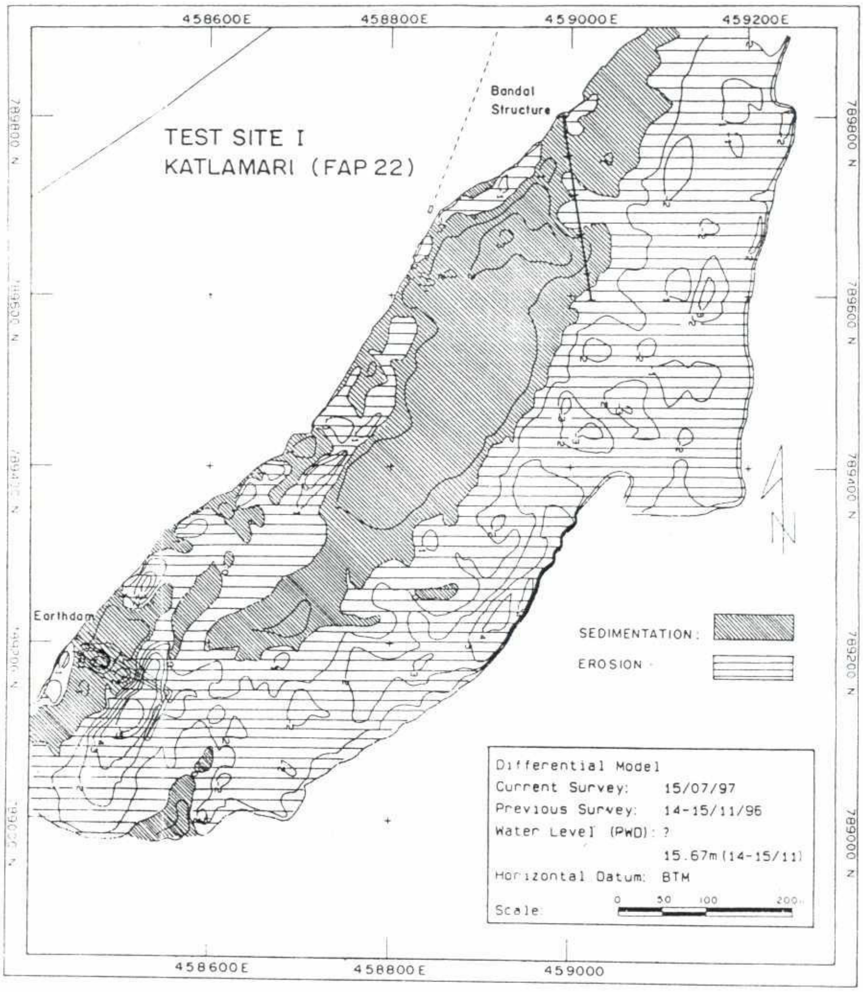
7.2.1 Bandals

(a) Positive

- The bandals survived the monsoon in good condition. This holds for all different Sections A through D. Only the Sections E1 and E2 were washed out in September when the deep part of the Fulchari channel reached the bandal tip. Also the substructure was in good condition, as could be verified during low waters (inspection January 1998);
- The structure remained intact in spite of the considerable scour hole, which was deeper than anticipated (5 instead of 4 m). It should be noted that the flows were according to the design values and the wave height was less than anticipated;
- The screen levels chosen seemed adequate during the very high water of mid July. No excessive extra scour occurred while the bandals were overtopped by about 0.8 m;
- The gradual reduction of the screen levels and height at the tip of the bandal structure appeared to work well. No excessive scour was generated there;
- The anchoring system of Section C remained intact. During a short period the bandal scour extended to the downstream pile row but did not destabilize part of Section C;
- From the various bandal designs Sections B and C were most successful. Bandals with adjustable screens should be further developed based on the B-type;
- Although the Fulchari channel changed considerably, especially during the second half of the 1997 monsoon and the deeper part of the channel shifted towards the Katlamari char (the test site) the main part of the char was not eroded;
- The result of the bandalling was impressive. Behind the bandals a char developed of about 150 x 500 m with 3 m of sedimentation as a maximum (see Fig. 7.1). This char protected part of the high right bank. Also just upstream from the bandal sedimentation on the char occurred.

(b) Negative

- The scour depth was more than estimated: 5 instead of 4 m. This is considerable also in view of the applied pile length of 7.5 m. This also implies that the possibilities to reduce the size and cost of the substructure are limited.
- Some limited bank scour was generated between the bandal and the high bank.
- The whole bandal was too short to close the Katlamari channel. The limited length was accepted because of various reasons
  - the target of testing in 1996/1997 was to obtain measurable effects;
  - extension implied construction in deeper water (construction from the river, larger structures), and
  - financial aspects.
- The prestressed ropes of Section C caught quite some floating debris, causing horizontal drag forces parallel to the screen, resulting in minor damages.
- Due to the horizontal shifting of part of the screen of Section C an opening occurred between the B and C screens. The considerable velocities through such an opening generated probably additional scour. It was there that the maximum scour was measured.



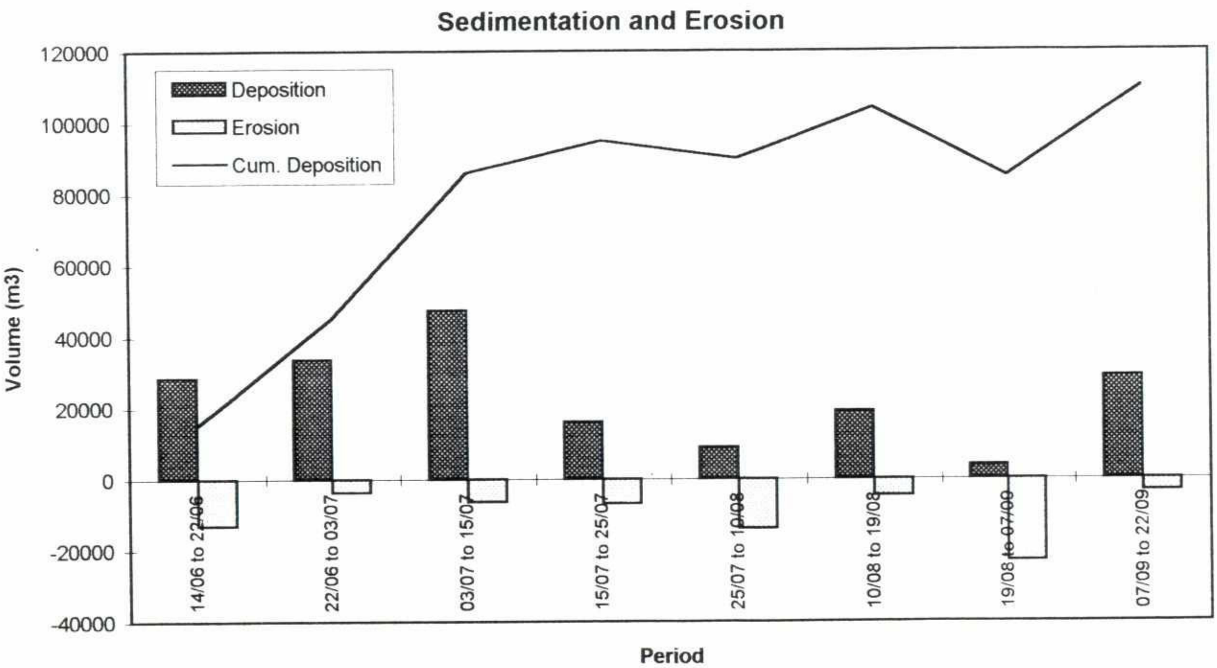


Fig. 7.1: Sedimentation behind bandals

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- In September increasing depth of the Fulchari channel in front of the bandals was observed. The edge of the char on which the bandals have been constructed was eroding. The Sections E1 and E2 were washed out. The flow and sediment concentration in the Katlamari channel were increasing.

7.2.2 Earth Dam

(a) Positive

- The design appeared successful in the main sections of the dam. This means that the structure of sand, geotextile and plastic gunny bags is appropriate;
- The dimensioning seems adequate (upstream slope 1:3, crest width 2 m, downstream slope 1.4). Lower crest levels (e.g. 16.00 m instead of 18.00 m+PWD) seem recommendable;
- Construction without compaction is possible although settlement up to 20% should be taken into account, and
- Only limited bed protection seems appropriate in the deepest part of the cross-section.

(b) Negative

- The connection between the dam and the banks and shallow channel parts appeared to be valuable and needs special measures to prevent/reduce erosion;
- The bypass could develop rapidly because of the insufficiently protected (light sill, no catkin) low laying excavated area on the char adjacent to the dam. The full hydraulic head, generating considerable flow velocities, was eroding the area, before the dam was overtopped. With a lower dam crest and/or a higher char area the problems would have been reduced. Then the (well protected) dam would have taken the flow first, reducing the head by which the velocities over the char would have been less violent. But also then the problems, although to a less extent, remain;
- The price of the dam appeared to be higher than expected. Half of the costs were spent on the gunny bags;
- The impact of the dam on the channel morphology was reduced as a consequence of the generated bypass channel;
- During the falling limb the downstream end of the Katlamari channel fell dry first. The flow turned and the remaining water volume discharged via the bypass around the dam and the scour channel under the bandals, substantially affecting the morphology;
- The expected sedimentation upstream of the dam did not occur. Partly because of the flow through the bypass, but also because of low sediment concentrations upstream of the dam. This is a consequence of the effects of the upstream bandals;
- Gunny bags are easily damaged especially the jute ones. They do not last for one full monsoon season, and
- The applied method of positioning of the bags on the upstream slope is less appropriate in waves.

7.2.3 Floating Screen

(a) Positive

- Probably the strongests three points in the concept are that
  - both local bandal scour and river erosion are not threatening the safety of the construction;
  - the high flexibility enables to adjust screen positions even in flood conditions for instance to counteract undesired morphological developments, and
  - the system is recurrent in the sense that it can be used several times at different locations.
- The principles of making anchoring systems from a string of concrete blocks appeared to work well also from operational point of view.

(b) Negative

- Country boats appeared not to be the right floating elements for the surface screens. Especially during screen operations (per screen section) torsion tended to damage the boats;
- Because of the geometry of boats and screens, considerable gaps between the screens were unavoidable, reducing the effectivity of the floating screens;
- Due to the bullah frame, keeping the boats rigidly together, many openings in-between the screen sections were unavoidable. Also this makes it impossible to have a more or less continuous screen;
- The renting costs of the big country boats were considerable;
- The country boat/screen configuration made it difficult to clean the system from floating debris;
- The anchoring system is probably the most critical part of the system. Self-burying systems have the advantage of increasing holding power, but may not be recoverable after some time.

7.3 POSSIBLE IMPROVEMENTS

7.3.1 Preliminary Remark

From the positive and negative experiences described above, some conclusions can be drawn regarding the design of the recurrent measures. Possible design improvements are suggested per structure.

7.3.1 Bandal

- Steel pipes are still to be considered especially for the substructure of the larger bandals (present size is to be considered a maximum in wood and/or bamboo);
- Adjustable screens seem attractive to improve the effectivity of the bandals. The screen operation can also be used to limit scour depth;
- The shape of the underside of the bandal may be adjusted to reduce scour depths;
- The velocities in the developed scour hole are relatively low (0.4 to 0.7 m/s) so there is certainly scope for reduction of the scour depth by means of a bed protection for instance made of gunny bags on a geotextile;
- The connection between the bandals and the bank need protection against erosion;
- Ropes of Section C to be placed lower in view of floating debris;
- The horizontal interconnections between screens, especially between bandal sections need to be strengthened;
- The horizontal stability in the main centre line of the bandals need to be revised;
- The idea of a floating line to divert floating debris deserves some follow up.

7.3.2 Dam

The main improvements in the design of the dam are:

- The dam crest should be made lower than the adjacent chars;
- The dam crest should be made lower in the deepest parts of the channel and higher near the edges;
- The design should be such that a considerable flow over the dam occurs before the unprotected surrounding char areas are flooded;
- The connection between dam and channel banks need considerable protection against erosion;
- The downstream channel bed behind the dam need more protection against erosion, especially in the shallower parts of the cross-section (where the high currents over the crest hit the channel bed);
- The position of dam in the channel need to be reconsidered. A dam downstream causes backflow during the receding limb of the hydrograph which may have serious morphological consequences;
- As dam failure is common, two or three dams in a row are recommended.

7.3.3 Floating Screen

- The country boats are less suitable to be used as floats. Better type of floats need to be designed;
- The screen dimensions and float length should be made such that a closed screen can be constructed;
- The screen should be made such that they can be shackled together so that a flexible chain of screens can be made;
- The shape should promote easy cleaning from floating debris;
- The anchoring system need further systematical development resulting in design anchor weight/holding power ratios;
- The anchoring system should be made such that when one anchor fails and a screen shifts the whole system does not collapse in a chain reaction.

7.3.4 Suggestions for Monitoring

The bathymetric surveys as done were very adequate in terms of frequency, line spacing and covered area. However, the related surveys of bankline, waterline and topography of chars did not match the timing of the bathymetric surveys. A better timing of these surveys will improve the consistency of the datasets for mapping. As soon as chars become visible in the falling limb, a waterline survey should be executed.

The positioning during scour depth measurements appeared often difficult resulting in uncertainties in the dataset. Attention need to be paid to the method of positioning to overcome the uncertainties.

The flow velocity measurements in the scour hole yielded essential information for the analysis. These measurements need to be extended to the full water depth and several verticals in the flow direction to obtain a more complete picture of the flow around the bandals (or other surface screens). As these flows are essentially three-dimensional the measurements should include the flow direction.

For the determination of hydraulic loads on the floating screens, and the holding power of anchoring systems, measuring of cable forces is a must. Systematic tests measuring forces as a function of flow direction and screen depth will give important information for the design of surface screens.

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92

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